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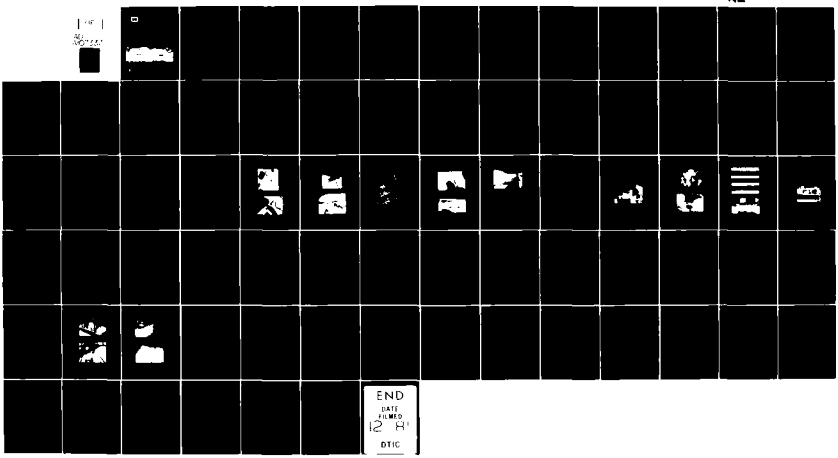

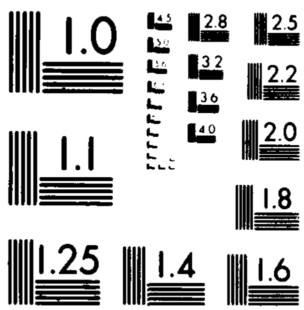

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MISCELLANEOUS PAPER SL-81-27

STRUCTURAL STABILITY EVALUATION WINNIBIGOSHISH DAM

by

Carl E. Pace

Structures Laboratory
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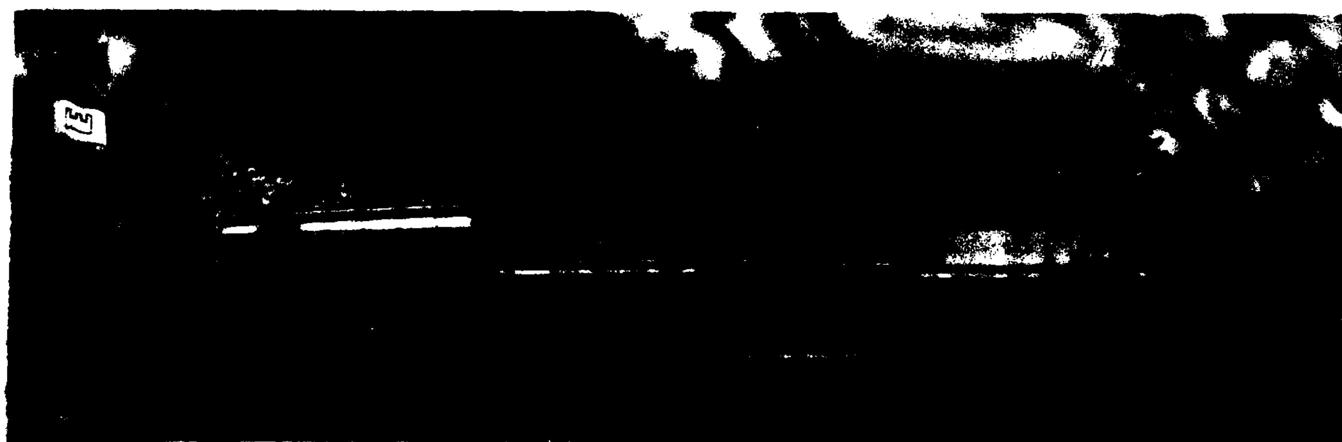
September 1981

Final Report



Prepared for U. S. Army Engineer District, St. Paul
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A typical interior monolith of Winnibigoshish Dam was evaluated for adequacy in stability for five load cases:

(1) Normal operation
(2) Normal operation with truck loading (H15-44)
(3) Normal operation with earthquake
(4) Normal operation with ice
(5) High-water condition.

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20. ABSTRACT (Continued).

By using the conventional stability analysis (rigid body assumptions), the approximate magnitude of loads which are on the foundation piles were determined; but, without knowing the supporting characteristics of the foundation material, an estimation of the adequacy of the structural stability was inconclusive.

The supporting characteristics of the foundation material were determined by in situ testing using a pressuremeter to predict the horizontal supporting characteristics of the pile-soil system.

Two NX core holes were drilled through typical monoliths to obtain access to the foundation material. One other hole was drilled to sample the supporting decking, beams, and piling. The pressuremeter tests were performed and in situ soils data obtained. The horizontal soil modulus was obtained as a variation with depth into the foundation material and with soil deformation for three test positions in each of the two test holes.

The horizontal modulus of subgrade reaction was then used in a three-dimensional direct stiffness analysis to determine the forces and deflections of the foundation piles. A beam on an elastic foundation analysis was performed and the pressure, moment, and deflection along the length of the most critically loaded pile were determined.

The axial or compressive loads were below allowables. The shear loads at the top of the piles are excessive, and it is recommended that 135 kips of strut resistance per small pier be provided in case it is required. The pile deflections are not excessive. The piles are not overstressed in flexure.

The average unconfined compressive strength of concrete exceeded 7000 psi, which is adequate for this type of structure. The concrete is well consolidated with some alkali-silica reaction. The alkali-silica reaction is not considered significant.

The deteriorated surface of the concrete and steel should be repaired (Part VI). After surface concrete and steel deterioration is repaired and strut resistance is provided for pier stability, the dam can be expected to perform satisfactorily for many more years.

PREFACE

The evaluation of the stability of Winnibigoshish Dam was conducted for the U. S. Army Engineer District, St. Paul, by the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES). Authorization for this investigation was given in Intra-Army Order for reimbursable services No. NCS-1A-78-75, dated 23 July 1979.

The contract was monitored by the U. S. Army Engineer District, St. Paul, with principal assistance from Messrs. Roger Ronning and Jerry Blomker. Their cooperation and assistance were greatly appreciated.

The study was performed under the direction of Messrs. Bryant Mather, William Flathau, and John Scanlon, SL. The structural stability analysis was performed by Dr. Carl Pace and Mr. Roy Campbell. The core logging and writing of the petrographic report was performed under the technical supervision of Mr. Alan Buck by Miss Barbara Pavlov and Mr. Sam Wong. The testing was performed by Mr. Mike Lloyd. The computer programming by Miss Alberta Wade was appreciated. The core drilling was under the direction of Mr. Mark Vispi, Geotechnical Laboratory, WES. Dr. Pace prepared this report.

Commanders and Directors during the conduct of the program and the preparation and publication of the report were COL John L. Cannon, CE, and COL Tilford C. Creel, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acre-feet	1233.489	cubic metres
cubic inches	0.00001638706	cubic metres
feet	0.3048	metres
inches	0.0254	metres
kips (force)	4448.222	newtons
kips · feet	1355.818	newton · metre
kips (force) per square foot	47.88026	kilopascals
miles (U. S. statute)	1609.344	metres
pounds (force)	4.448222	newtons
pounds (mass)	0.4535924	kilograms
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per inch	175.1268	newtons per metre
square miles (U. S. statute)	2.589988	square kilometres

STRUCTURAL STABILITY EVALUATION,

WINNIBIGOSHISH DAM

PART I: INTRODUCTION

Background

1. Winnibigoshish Dam (Figures 1 and 2) is located on the Mississippi River, 1247.9 river miles* above the mouth of the Ohio River. The dam is at the outlet of Winnibigoshish Lake, which is about 14 miles northwest of Deer River, Minnesota, in the southwest portion of Itasca County. It is approximately 170 river miles downstream from the source of the Mississippi River at Lake Itasca, and 408 river miles above St. Paul, Minn.

2. The total drainage area above the dam is 1442 square miles. At the operating stage of 15.34 ft, the reservoir (Figure 3) has an area of approximately 179.4 square miles.

3. The primary purpose for Winnibigoshish Reservoir was for storing water to improve navigation on the Mississippi River between St. Paul and Lake Pepin. Demands by resort and private property owners in the reservoir area have resulted in revised regulations that have reduced the usable storage capacity of the reservoir by limiting its drawdown. During periods of abnormally high inflow, storage is utilized up to the 15.34-ft stage. Flowage rights have been acquired to an 19.14-ft stage to allow for wave action and seepage damage. To provide storage capacity for the spring runoff, the reservoir is lowered during the winter months to reach a 9.14-ft stage by April 1. General data for the reservoir are presented in Table 1.

4. Winnibigoshish Dam construction was begun in 1882 and completed in 1891. It was the first of the Mississippi River headwater reservoirs to be built, and served as a pilot project for the rest of the headwaters

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* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 3.

system. The original construction of the dam was a timber crib and piling structure. The embankment and dikes were earth fill with a timber diaphragm core filled with puddled clay that was protected by sod and riprap surfacing. The lake side surface of the 800-ft-long control structure dike is protected by grouted riprap that runs from the control structure to the right bank. The top of the 800-ft dike supports a 20-ft roadway.

5. Beginning in 1899, the timber crib superstructure was removed and a new concrete superstructure was constructed in its place, with reconstruction completed in 1901. A steel bridge was constructed across the structure in 1909, with subsequent replacement in 1934. A fishway was constructed landward of the left abutment in 1912-1914. The tainter gates, bear trap gate, and operating machinery were removed in 1931, and operation from that time until 1966 was by stop log only. In 1966, five of the stop logs bays were fitted with steel slide gates. Considerable repairs were needed from time to time on the timber aprons, and in 1964-1966 a concrete apron was constructed. The embedment and perimeter dikes were raised and strengthened, and the riprap was repaired on several occasions during the early life of the project. The upper operating limit of the reservoir has been exceeded twice (in 1905 and 1950), but the flowage limit has never been exceeded. General data concerning the dam are presented in Table 2. More detailed information can be obtained from reports by the U. S. Army Engineer District, St. Paul (1973 and 1977).

Control Structure

6. The control structure consists of reinforced concrete abutments and piers, supported on timber piling. There are five 14-ft sluiceways, each of which is divided into three sections of stop logs. In 1966, a 3-1/2- by 5-ft slide gate was added to each sluiceway (see Figure 4). In addition, there are a 12-ft log sluice and a 5-ft fishway (no longer used) in the structure. The total length between abutments is 162 ft. The control structure also supports a 20-ft highway bridge. A view of

the dam is presented in Figure 5, and an additional view showing the top of the piers and the bridge superstructure is presented in Figure 6.

Objective

7. The objective of this study is to evaluate the stability of the concrete control structure.

Scope

8. The study was limited to a structural stability evaluation of the concrete control structure with consideration given to foundation and concrete properties. To aid in this evaluation, two cores were drilled through the dam into the foundation. The foundation material was tested in situ in order to determine its supporting capabilities in relation to the structural piling. The cores and foundation material were examined and the structural stability of the dam was evaluated. The stability analysis was performed in accordance with current Corps of Engineers criteria.

PART II: CORING PROGRAM

9. Since Winnibigoshish Dam falls into the classification of a low-head dam, a limited coring program was performed to obtain properties of the concrete and to obtain access to the foundation material for in situ testing.

10. Three NX concrete cores were obtained. Two cores were obtained from pier 1 and one from pier 3. The piers are numbered from right to left looking from upstream to downstream. The locations of the core holes in piers 1 and 3 are presented in Figure 7 along with a schematic of a typical pier and roadway of Winnibigoshish Dam. One of the cores through pier 1 sampled the timber supports and a foundation pile and found that both the timber and the pile were in excellent condition. The second core hole in pier 1 and the core hole in pier 3 were used to reach the foundation material and test it for in situ properties.

11. Cores in pier 1 were obtained by building and placing a platform (Figure 8) spanning piers 1 and 2 on which the drilling rig was mounted. A crane had to be used to place and remove the drilling rig. The core hole in pier 3 was drilled by using a truck-mounted drilling rig to core through the roadway and pier.

12. Diamond core bits and 5-ft-long double-tube swivel-head core barrels were used to obtain cores from the concrete. A slotted casing was used to house a pressuremeter probe as the casing was driven to the desired depth for pressure tests.

13. The coring program was oriented toward determining:

- a. Depth of deteriorated concrete
- b. Uniformity of concrete with depth
- c. Unconfined compressive strength of the concrete, and
- d. To provide access to the foundation in order to obtain in situ tests and properties of the foundation.

14. The in situ strength of the foundation material was an important factor in the analysis of the stability of the dam, which is supported on timber piles embedded in the foundation material. The drilling rig was used to perform pressuremeter tests, standard penetration tests, and to obtain disturbed samples of the foundation material.

15. The coring program was considered a minimum for obtaining representative information on the concrete and foundation material but was adequate for this particular dam. The core holes were not grouted; instead caps (Figure 9) were used to seal the openings so that the core holes could be used in the future for obtaining piezometric data.

16. Pictures of representative concrete cores are presented in Figure 10. A close view of a core and cut section is presented in Figure 11. The concrete at Winnibigoshish Dam is very uniform.

PART III: PETROGRAPHIC REPORT AND CORE LOGS

Samples

17. Three NX concrete cores were received at the U. S. Army Engineer Waterways Experiment Station on 29 October 1979 for tests and examination. The cores were taken from two piers at Winnibigoshish Dam. All cores were from vertical borings. The cores are described and identified below:

Core No.	Elevation of Top* of Core	Length	Location	Material
W-P1A	1294.96 ft	12.4 ft	Pier 1	Concrete
W-P1	1294.96 ft	14.5 ft	Pier 1	Concrete, wood
W-P3	1294.96 ft	13.2 ft	Pier 3	Concrete, wood

Test Procedure

18. All of the material shipped from the three holes was examined and logged. The appearance of the concrete was noted, and representative pieces of the cores that contained visual evidence of poorer quality concrete or significant reaction products, as well as pieces of typical concrete, were selected for more detailed examination. Representative samples were also selected for physical tests. Samples of sufficient length were selected from the top, middle, and bottom portions of cores for unconfined compression tests.

19. A length of core was chosen to represent the general overall quality of the concrete in the dam. This piece of core was sawed longitudinally, and a sawed surface was then ground smooth. The ground surface of the concrete was examined with a stereomicroscope.

20. Cement paste concentrate was prepared and examined by X-ray diffraction. This was obtained from a sample of typical intact concrete. The paste was concentrated by gentle crushing of the concrete and passing the fragments over a No. 100 sieve. The material passing this sieve was

* All elevations are referred to msl, 1929 adjustment.

then ground to pass a No. 325 sieve, backpacked to minimize orientation, and examined by X-ray diffraction. All X-ray patterns were made with an X-ray diffractometer using nickel-filtered copper radiation.

21. Samples of the white reaction product found in some voids and coating some aggregate surfaces in the concrete were examined using a stereomicroscope and as immersion mounts using a polarizing microscope.

Results

22. The general condition of the concrete from each core is described in the following paragraphs:

Cores W-P1 and W-P1A

23. These cores came from pier 1. The concrete in both cores was similar and consisted of well consolidated, nonair-entrained concrete (Figures 12 and 13). While maximum aggregate size was judged to be about 2 in., there was what looked like a 6-in. cobble in core W-P1 at the 11.44-ft depth. The coarse and fine aggregate appeared to be natural gravel and sand; some crushing of the gravel may have been done to improve the grading. The coarse aggregate and the larger sand sizes were particles of granite, granite gneiss, and fine-grained dark igneous rock.

24. The finer sand sizes appeared to be largely individual grains of quartz or feldspar. Breaks in the cores were generally believed to be caused by the drilling operation, although the break at about 1.2 ft in core W-P1 was coated with alkali-silica gel and may have been an old break.

25. The presence of white alkali-silica gel in some voids and on some aggregate surfaces showed that some alkali-silica reaction had occurred. As indicated in Figure 12, some wood from decking, beams, and piling beneath the concrete was included with core W-P1.

26. The first 5.5 ft of core W-P1A (Figure 13) contained breaks every 0.1 to 0.2 ft, while breaks in core W-P1 occurred approximately every foot. Slight honeycombing was found in both cores at approximately 1.2 to 1.4 ft and 10.6 ft.

Core W-P3

27. This core came from pier 3 and contained concrete similar to that found in the two previously mentioned cores (Figure 14). Maximum aggregate size was about 1-1/2 in. Aside from this slight difference in top size, the aggregates were the same. Breaks in the core were new and were probably caused by the drilling operation. Alkali-silica gel indicated reaction as in cores W-P1 and W-P1A. The bottom of core W-P3 included both wood decking and intact porous grout. The grout represented material placed at a later dewatering after construction was completed.

28. The broken surfaces of several coarse aggregate particles, especially the granite and granite gneiss, contained thin rims that may have been caused by the alkali-silica reaction.

29. The alkali-silica reaction gel in these cores was a white to translucent, sugary-textured material filling voids or coating particles. In an oil immersion mount under plane-polarized light, the gel appeared as alternating layers of clear and brown translucent gel. Index of refraction for the gel was <1.498. Using crossed polarizers, there were two varieties of the gel reaction; one with a salt and pepper appearance, resembling chert, and the second with low birefringence, wavy extinction, and layering.

30. The X-ray diffraction pattern of the cement paste concentrate revealed crystalline compounds which are normal constituents of hydrated portland cement paste. These inclued ettringite, calcium hydroxide, and tetracalcium aluminate carbonate-11-hydrate (monocarboaluminate). Necessary contamination from the aggregate showed quartz, plagioclase, and potassium feldspar, and calcite. The calcite may be from the aggregate, from carbonation of the cement paste, or a combination of both. The quartz and feldspars were expected, as they are usually major constituents of these aggregate rock types.

Discussion

31. All of the concrete was similar and in good physical condition. Fragmentation and breaks near the surface of core W-P1A were frequent. However, there was no other indication that this concrete was of lower quality than the concrete in the other cores examined. In general, the fractures in the cores appeared to be new and to have been produced by the drilling operation.

32. While there was evidence of alkali-silica reaction in each core, the condition of the concrete that was seen suggested that little or no damage had occurred because of this reaction. The presence of this reaction with aggregates long considered inert is interesting. It agrees with other reports (Buck and Mather 1978, Buck and Burkes 1978, and Buck 1978) of alkali-silica reaction with similar aggregates.

33. No damage due to frost action was recognized. Since the concrete was nonair-entrained, this means freezing of the concrete while saturated did not occur.

PART IV: FOUNDATION AND CONCRETE PROPERTIES

In Situ Foundation Testing

34. An estimation of the foundation supporting characteristics for an in-place structure supported by piling is usually based on the material properties derived from sampling the foundation material, transporting the sample to the laboratory, preparing the sample for testing, and testing the sample. The foundation properties obtained in this manner are at best an approximation. Soil conditions and stress fields can be controlled in the laboratory, but just how faithful these conditions represent in situ conditions is a matter of conjecture. A further complication at Winnibigoshish Dam was that due to a saturated sand and gravel foundation material the ability to obtain undisturbed samples was doubtful.

35. For the above reasons, it was considered best to test the foundation material in situ by determining the resistance of the soil to horizontal deformation. Hence, in situ testing of foundation material supporting the piling was accomplished to obtain the structural supporting characteristics of the pile-foundation system. The pressuremeter method was used to measure deformation properties and obtain a rupture or limit resistance of the foundation material.

Pressuremeter Field Tests and Results

36. In order to test the material that supports the pile foundation under Winnibigoshish Dam, an access to the foundation material had to be obtained. This was done by coring NX holes through the dam piers and down to the foundation material. Then, a slotted casing that contained the pressuremeter probe was driven to the desired test elevation in the foundation material. A pressurized bottle of gas was used as the pressure source. The pressuremeter tests were performed at three elevations in hole W-P1A and in hole W-P3. The locations of the probe below the bottom of the pier are presented in Table 3 for test holes W-P1A and W-P3.

37. Standard penetration (split spoon) tests were performed in test holes W-P1A and W-P3 with the results presented in Table 4. The tests were only used to obtain an indication of the compaction of the upper material supporting the piling.

38. Disturbed samples of the foundation material were obtained and transported to the laboratory for classification tests. The foundation material supporting the structure and piles under Winnibigoshish Dam was found to consist mainly of sand and gravel, with some clay (Figures 15-23).

39. The main characteristic of the foundation material, which indicates its supporting capability for a pile foundation, is the subgrade modulus, and its variation with pressure and depth into the foundation. The pressuremeter tests were used to obtain these data. Plots of data for holes W-P1A and W-P3 are presented in Figures 24-30 and Figures 31-37, respectively.

40. The recorded pressure must be corrected to compensate for the hydrostatic pressure of water in the tubing and for the probe calibration, which gives the resistance to expanding of the rubber membrane. The corrected pressure curves are presented in Figures 24-26 for hole W-P1A and Figures 31-33 for hole W-P3.

41. Several methods were used in obtaining the limit pressure of the foundation material; but, for Winnibigoshish Dam, it was found that extrapolating the curves out to two times the initial volume of the cavity gave excellent results. The limit pressures are presented in Figures 25 and 32 for holes W-P1A and W-P3, respectively.

42. The shear modulus (Gabuelin, Jiziquel, and Shields 1978) depends not only on the slope of the pressure-volume curve but also on the volume of the probe. The average volume is used in calculating the shear modulus is as follows:

$$G_M = 535 + \frac{V(I) + V(I+1)}{2} \quad \frac{\Delta P}{\Delta v} \qquad (1)$$

$$= 535 + \frac{V(I) + V(I+1)}{2} \quad \frac{P(I+1) - P(I)}{V(I+1) - V(I)}$$

43. The deformation modulus, which is something roughly equivalent to Young's modulus, is obtained from the well-known relation:

$$G_M = \frac{E_p}{2(1 + v)} \qquad (2)$$

44. Poisson's ratio is used as 0.33, and the resulting deformation modulus is called the Ménard modulus, E_M.

$$E_M = 2(1 + v)G_M \qquad (3)$$

$$= 2(1 + 0.33)G_M = 2.66G_M$$

The Ménard modulus is presented in Figures 28 and 35 for hole W-P1A and W-P3, respectively.

45. The horizontal subgrade modulus (k) is obtained from the following equations:

$$\frac{1}{k} = \frac{2}{9E_M} \cdot B_o\left(\frac{B}{B_o} \times 2.65\right)^{\alpha} + \frac{\alpha}{6E_M} \cdot B \qquad (B > 0.6\ \mathrm{m}) \qquad (4)$$

$$\text{or} \quad \frac{1}{k} = \frac{B}{E_M}\left(\frac{4(2.65)^{\alpha} + 3\alpha}{18}\right) \qquad (B < 0.6\ \mathrm{m}) \qquad (5)$$

where

B_o = the reference pile diameter, 0.6 m

B = the pile diameter

α = rheological coefficient given in Figures 3-48 of Baguelin, Jiziquel, and Shields (1978).

46. After a representative value of k has been determined, it can be multiplied by the pile diameter to obtain the horizontal modulus of reaction for the pile-soil system. The horizontal modulus of reaction of the soil can be used in the piling analysis to obtain deflections, forces, and moments to use in evaluating the adequacy of the pile foundation.

Concrete and Piling

47. Hole W-P1 was primarily used to sample the supporting timber decking, beams, and piling under the pier. The timber was in excellent condition. The 12-in.-diam Norway Pine piling that supports the monoliths at Winnibigoshish Dam is approximately 15 ft long. The properties and allowables for the 12-in. Norway Pine piling are as follows:

Modulus of elasticity (E) = 1.32×10^6 psi

Shear modulus (G) = 0.45×10^6 psi

Allowable compressive stress parallel to grain = 1100 psi

Allowable tensile stress parallel to grain = 775 psi

Allowable average shear stress = 75 psi

Allowable compressive load on a pile = 124 kips

Allowable tensile load on a pile = 0 kips

Average allowable lateral load per pile = 8.5 kips

Allowable moment in a pile = 131,000 in.-lb or 10.9 kip-feet.

The properties of Norway wood can be found in many handbooks. One such reference is presented (Southern Pine Association, 1954).

48. The unconfined compressive strength of the concrete is presented in Table 5. The average unconfined compressive strength was 7050 psi, which is excellent. Since the interior concrete has performed so well over 75 years of service, if the surface concrete is repaired and kept from accelerated deterioration, the structure can be expected to perform satisfactorily for many more years.

49. The concrete surfaces (Figures 38 and 39) need rehabilitation to eliminate possible water entry into cracks and freezing, causing accelerated deterioration.

50. There are a number of methods of repair that might be used; but, the upper headwater structures are ideal for an economical repair program such as:

a. Clean surface concrete.

b. Fill cracks.

c. Paint on a cementitious coating to rehabilitate the surface concrete.

This type repair can be performed rapidly and economically. It is analogous to cleaning and filling cracks and then painting a room in a house. Local labor could do the work with only common tools.

51. Under some conditions, an acrylic-polymer coating of a composition as listed in Tables 6 and 7 might be used. Certain acrylic-polymers have exhibited good bond and noncracking characteristics when used in ordinary environments. They have also shown good resistance to freezing-and-thawing environments. The particular polymer to be used should be tested as follows before being used to rehabilitate the surface concrete of the Upper Mississippi Headwater Structures.

a. Determine the resistance of the coating to cracking during extreme temperature changes.

b. Determine its ability to retain bond capability in freezing-and-thawing environments.

c. Determine its ability to "breathe," thus allowing water to escape from the interior concrete through the coating, preventing critical saturation of the concrete.

PART V: STABILITY ANALYSIS

Conventional Stability Analysis

52. A conventional stability analysis assumes that the base of the structure is rigid and obtains the loads on the piles. The analysis does not consider the load redistribution due to the pile and structure deformations with consideration being given to the strength characteristics of the soil on the piling system. The monoliths at Winnibigoshish Dam were of such size and shape that the assumption of a rigid base was adequate. The supporting characteristics of the soil were taken into account by a modulus of subgrade reaction which was obtained from in situ testing of the foundation material (Part IV).

53. The geometry and loadings on a particular interior monolith at Winnibigoshish Dam are presented in Figure 40. The following five load cases were analyzed:

- a. Normal operation.
- b. Normal operation with truck loading (H15-44).
- c. Normal operation with earthquake.
- d. Normal operation with ice.
- e. High-water condition.

The applied loads and moments on the pile system are presented in Figures 40-45. A summary of the forces and moments on the piling system obtained by conventional stability analysis is presented in Table 8. At this point, the adequacy of the pile foundation could not be evaluated because the allowable vertical and horizontal loads had to be known to judge the adequacy of the piles. These allowables were not known for Winnibigoshish Dam.

54. To determine the adequacy of the stability of the pile foundation, in situ testing was performed to determine the supporting characteristics of the foundation material. The variation of modulus of subgrade reaction with depth and deformation was obtained. The modulus of subgrade reaction (Figures 30 and 37) was fairly constant with depth;

therefore, a conservative constant value ($4000 \frac{psi}{in.}$) was used in evaluating the pile-foundation system. This value was decreased to $1200 \frac{psi}{in.}$ due to close pile spacing. The reduction factor was calculated by the following formula as suggested by Davisson (1970).

$$h_a = 0.15 \frac{a}{B} - 0.2 \qquad (3 < \frac{a}{B} < 8)$$

where

h_a = reduction factor

a = center to center pile spacing from upstream to downstream

B = pile diameter

If the piling layout was adequate using this analysis, the total dam could be considered adequate in stability.

55. Determination of the allowable loads on the pile was based on the material properties of the pile. Evaluation of the stability of the pile foundation was based on the deflections at the top of the pile. If the horizontal deflections of the top of the pile were less than 1/4 in., the piling system would be considered adequate.

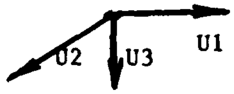
Pile Foundation Analysis Using In Situ Soil-Foundation Properties

56. A general, direct stiffness analysis for a three-dimensional pile foundation was used as has been presented by Saul (1968), which expands the Hrennikoff (1950) method from two dimensions to three. The general solution using this stiffness analysis follows.

57. The forces on a single pile can be equated to the pile displacements by the expression

$$\{F\}_i = \{b\}_i\{X\}_i \tag{6}$$

The $\{b\}_i$ values are the individual pile stiffness-influence coefficients, called the elastic pile constants. The positive system is as follows:



The $\{b\}_i$ matrix for a three-dimensional system can be defined for the i^{th} pile as

$$\{b\}_i = \begin{bmatrix} b_{11} & 0 & 0 & 0 & b_{15} & 0 \\ 0 & b_{22} & 0 & b_{24} & 0 & 0 \\ 0 & 0 & b_{33} & 0 & 0 & 0 \\ 0 & b_{42} & 0 & b_{44} & 0 & 0 \\ b_{51} & 0 & 0 & 0 & b_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & b_{66} \end{bmatrix}$$

The elastic pile constants are defined as follows:

- b_{11} is the force required to displace the pile head a unit distance along the U_1-axis, FORCE/LENGTH.
- b_{22} is the force required to displace the pile head a unit distance along the U_2-axis, FORCE/LENGTH.
- b_{33} is the force required to displace the pile head a unit distance along the U_3-axis, FORCE/LENGTH.
- b_{44} is the moment required to displace the pile head a unit rotation around the U_1-axis, FORCE-LENGTH/RADIAN.
- b_{55} is the moment required to displace the pile head a unit rotation around the U_2-axis, FORCE-LENGTH/RADIAN.
- b_{66} is the torque required to displace the pile head a unit rotation around the U_3-axis, FORCE/RADIAN.
- b_{15} is the force along the U_1-axis caused by a unit rotation of the pile head around the U_2-axis, FORCE/RADIAN.
- $-b_{24}$ is the force along the U_2-axis caused by a unit rotation of the pile head around the U_1-axis, FORCE/RADIAN. (Note: The sign is negative.)
- b_{51} is the moment around the U_2-axis caused by a unit of displacement of the pile head along the U_1-axis, FORCE-LENGTH/LENGTH.
- $-b_{42}$ is the moment around the U_1-axis caused by a unit displacement of the pile head along the U_2-axis, FORCE-LENGTH/LENGTH. (Note: The sign is negative.)

58. Pile i may be located in the foundation with axes through its origin parallel to the foundation axes. The foundation loads $\{Q\}$ and displacements $\{\Delta\}$ are located with respect to the foundation axes.

59. The forces $\{F\}_i$, due to the pile loads on the pile cap, are in equilibrium with a set of forces $\{q\}_i$ at the coordinate center of the pile cap. Equilibrium yields:

$$\{q\}_i = \{c\}_i\{F'\}_i \tag{7}$$

in which $\{c\}_i$, the statics matrix for a three-dimensional system, is

$$\{c\}_i = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & -u_3 & u_2 & 1 & 0 & 0 \\ u_3 & 0 & -u_1 & 0 & 1 & 0 \\ -u_2 & u_1 & 0 & 0 & 0 & 1 \end{bmatrix}$$

where

$u_1 = U_1$ coordinate of the pile, LENGTH.
$u_2 = U_2$ coordinate of the pile, LENGTH.
$u_3 = U_3$ coordinate of the pile, LENGTH.

Foundation Stiffness Analysis

60. If the piling cap is assumed rigid, then the deflection of the pile cap can be related to the deflection of the piling in the foundation axis coordinates by

$$\{x'\}_i = \{c\}_i^T \{\Delta\} \tag{8}$$

61. The foundation load $\{Q\}$ is distributed to each piling so that

$$\{Q\} = \sum_{i=1}^{n} \{q\}_i \tag{9}$$

where n = number of piles. The relationships between the foundation load and the pile cap deflections are

$$\{Q\} = \{S\}\{\Delta\} \tag{10}$$

in which $\{S\}$ is the stiffness-influence coefficients matrix for the foundation as a whole. The $\{S\}$ matrix is found by introducing the contribution of each individual pile toward the stiffness of the pile cap. This yields

$$\{q\}_i = \{S'\}_i\{\Delta\} \tag{11}$$

in which

$$\{S'\}_i = \{c\}_i\{a\}_i\{b\}_i\{a\}_i^T\{c\}_i^T \tag{12}$$

and finally

$$\{S\} = \sum_{i=1}^{n} \{S'\}_i \tag{13}$$

where $\{a\}_i$ is the transformation matrix of force and displacement of the pile (rotated and/or battered) axis to the foundation axis.

62. Once the stiffness matrix is known for the total foundation, the problem is essentially solved and only requires back substitution to find the distribution of loads to the individual piling. It can be noted that the foundation-stiffness matrix $\{S\}$ is independent of the external loads.

Loads and Displacements

63. The displacements of the pile cap can be found by inverting the foundation-stiffness matrix $\{S\}$ and multiplying it by the external load matrix $\{Q\}$ or,

$$\{\Delta\} = \{s\}^{-1}\{Q\} \tag{14}$$

64. Once the foundation deflections are known, the deflection of pile i about its own axes can be found by

$$\{x\}_i = \{a\}_i^T\{c\}_i^T\{\Delta\} \tag{15}$$

65. Finally, the forces allotted to each pile about its axes can be found from Equation 6 where

$$\{F\}_i = \{b\}_i \{x\}_i \qquad (6)$$

Forces and Deflections of Individual Piles

66. The approach followed in obtaining the forces and deflections on the individual pile is as follows. The modulus of subgrade reaction, the material properties of the pile, and the pile length are used to determine the pile-head stiffness matrix for a single pile, assuming a linear elastic pile-soil system. This pile head-stiffness matrix was obtained by using a finite element computer code (Marlin, Jones, and Radhakrishnan) which is a one-dimensional finite element analysis of a beam on an elastic foundation.

67. The pile-head stiffness matrix was then used as input in another computer program that uses the direct stiffness analysis to obtain the forces and deflections of the piles. A beam on elastic foundation analysis is then performed and the pressure, moment, and deflection along the length of the most axially loaded pile is determined (Figure 47).

68. The analysis assumes that the top of the pile is pinned to the base of the monolith, and the monolith base is rigid. These assumptions were adequate for the dam construction at Winnibigoshish Dam.

69. The results of the three-dimensional direct-stiffness pile-foundation analysis are presented in Table 9. The allowable loads on the pile were as follows:

$$\text{Maximum allowable compressive load} = \frac{(1100 \text{ psi})(3.14)(6)^2}{1000} = 124 \text{ kips}$$

$$\text{Maximum allowable shear load} = \frac{(75 \text{ psi})(3.14)(6)^2}{1000} = 8.5 \text{ kips}$$

Maximum allowable tensile load on a pile = 0 kips

70. The axial, or compressive loading, were well below the allowable based on the strength of the pile. The shear loads were above allowables. For normal operation plus ice loading, the average shear load was 12.5 kips/pile in relation to the allowable of 8.5 kips/pile. Ice loading has never been a real problem at Winnibigoshish Dam, but the shear loading is such that it is recommended to perform remedial measures such as providing for the development of 135 kips of strut resistance per pier downstream of the dam. This capacity for strut resistance can

be obtained by blocks secured with soil anchors in front of the pier or by assuring positive strut resistance from the pier to the downstream apron. The conventional stability analysis obtains pile loads which compare well with those obtained by the direct stiffness analysis.

71. The horizontal and vertical deflections of the piles were below the allowable of one-quarter inch and are acceptable. Since there are no noticeable vertical deflections of the structure and due to the adequacy of the foundation, it is not expected that vertical deflections of the dam monoliths will cause any loss of reservoir pool; therefore, the vertical deflections of piles were computed only as axial deflections $(\frac{PL}{AE})$ to save time and expense.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Foundation

72. The soil-piling system that supports the piers at Winnibigoshish Dam is adequate except for the safety of the piles in their resistance to shear loads. It is recommended that 135 kips of strut resistance be assured downstream of each interior small pier. The foundation material is adequate in supporting characteristics. The wood decking, supporting beams, and piling have been continuously submerged and from samples of the material from core hole W-P1 it was determined that the wood is in excellent condition.

Concrete and Steel

73. The interior concrete is of good quality, but the concrete and steel surfaces are deteriorating. The concrete surface needs to be rehabilitated to insure that water is not allowed to enter cracks and accelerate the deterioration of the concrete.

74. There are a number of methods of rehabilitation repair that might be used. However, the acrylic-polymer coating repair method, as previously described in Part IV, may be the most ideal. An acrylic-polymer coating selected under the guidelines set in Part IV should resist cracking, yet retain good bond capabilities under extreme temperature changes. The coating should also allow water within the concrete to "breathe" out through the coating so as to prevent development of critical saturation in the concrete. Since the concrete is not air-entrained, it cannot tolerate freezing while saturated. This fact may be sufficient to contraindicate the use of any coating based on organic plastics and adhesives.

75. The steel surfaces also need to be cleaned and painted.

76. After the rehabilitation has been accomplished, the structure should be adequate for many more years of service.

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__________ . 1977. "Mississippi River, Headwaters Reservoirs, Master Plan for Public Use Development and Resource Management," St. Paul, Minn.

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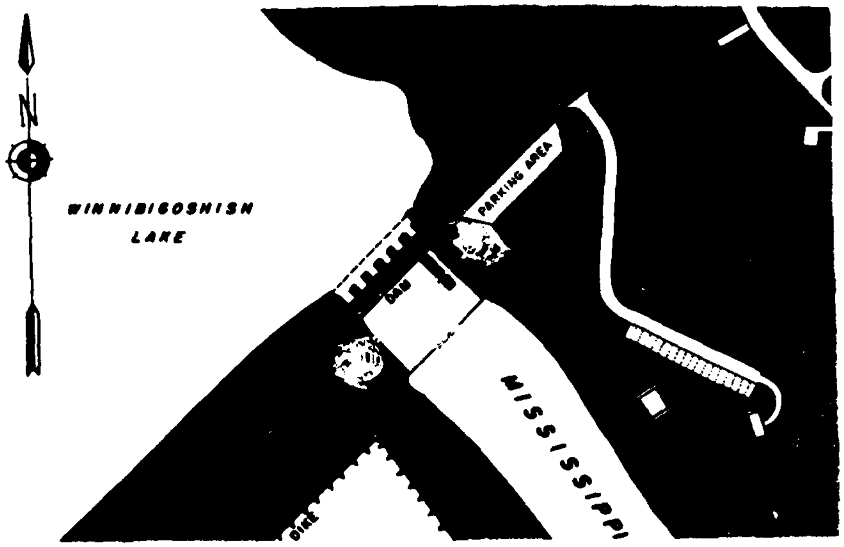
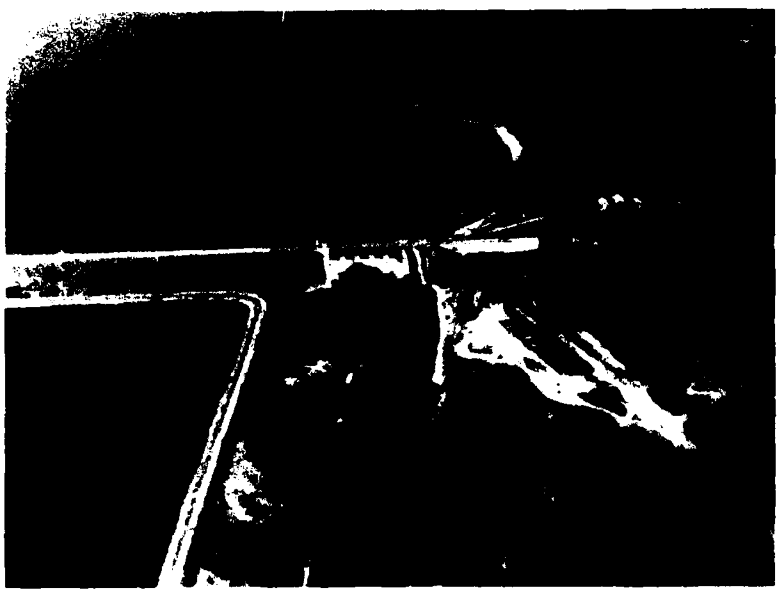
a. Vicinity map



b. Plan view at dam site

Figure 1. Area maps of Winnibigoshish Dam

a. Aerial site photo



b. Aerial photo of control structure and discharge channel

Figure 2. Aerial photos (taken 19 May 1972)

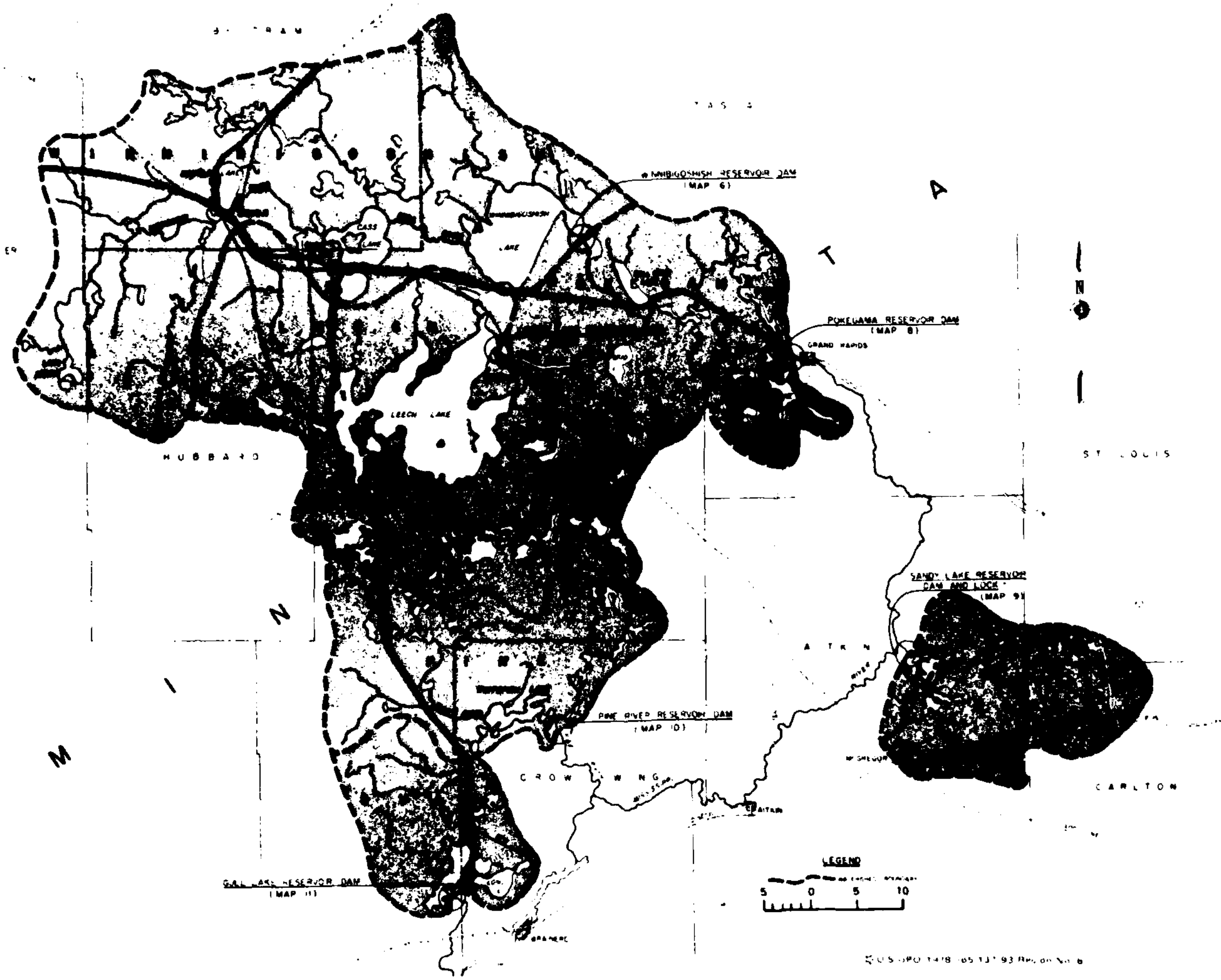


Figure 3. Project map of Mississippi River Headwaters reservoirs

Figure 4. Slide gate allowing water discharge

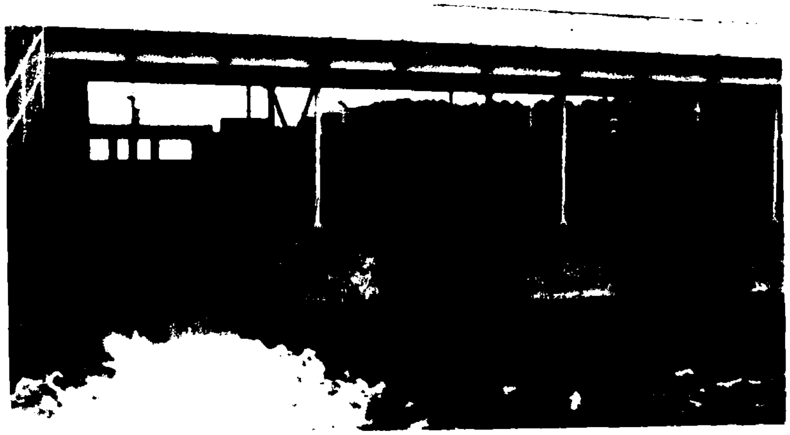


Figure 5. View of control structure, looking upstream from downstream end of right abutment extension

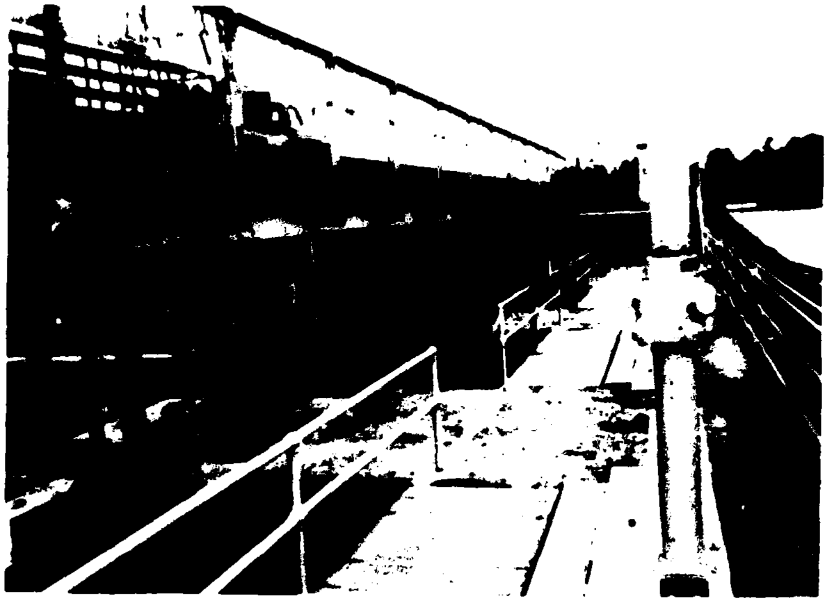


Figure 6. View of top of dam piers and roadway superstructure

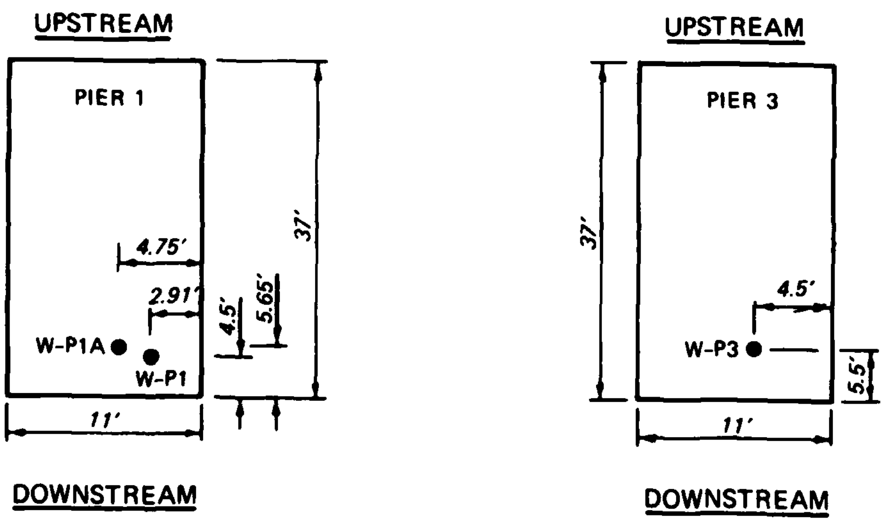


Figure 7a. Schematic plan view presenting locations of core holes in piers 1 and 3

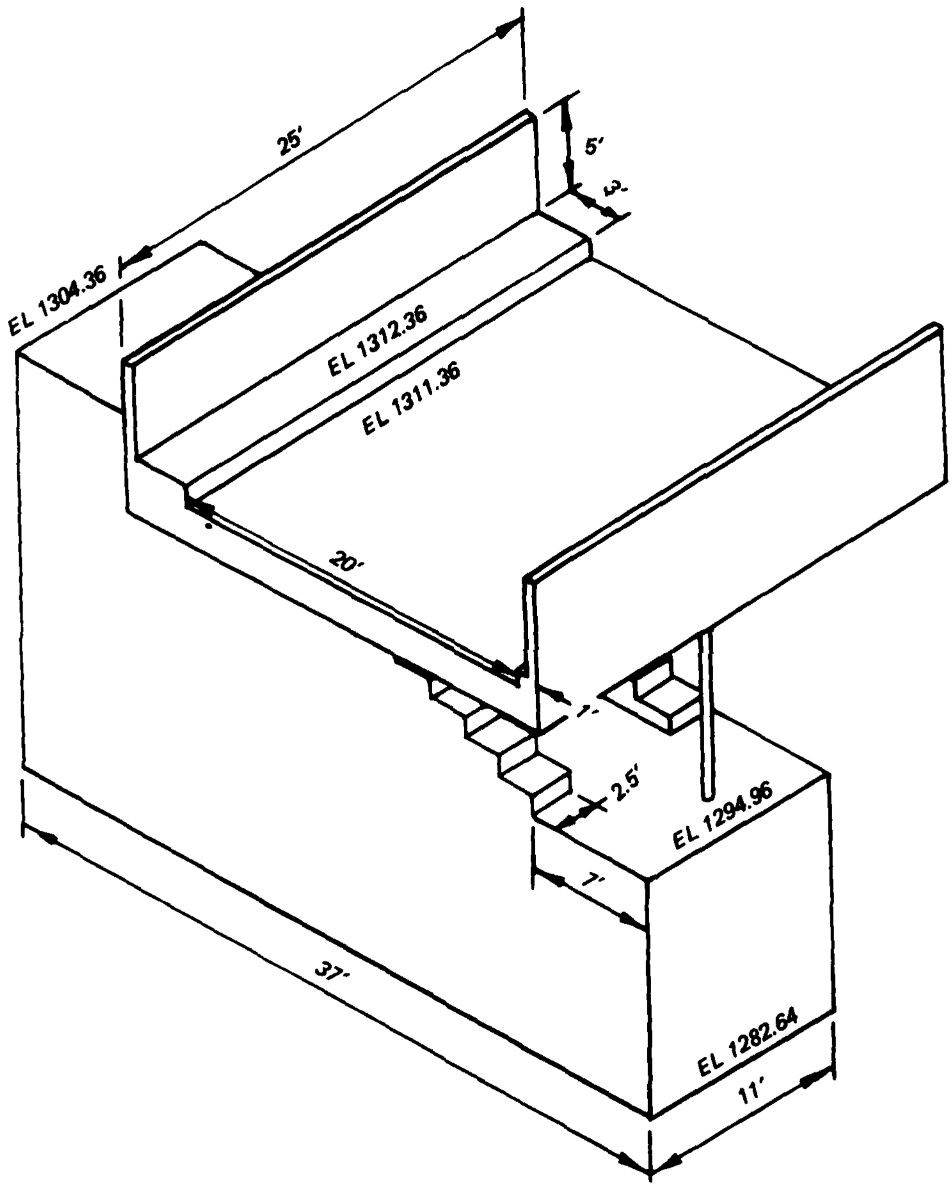


Figure 7b. Schematic of typical pier and roadway, Winnibigoshish Dam

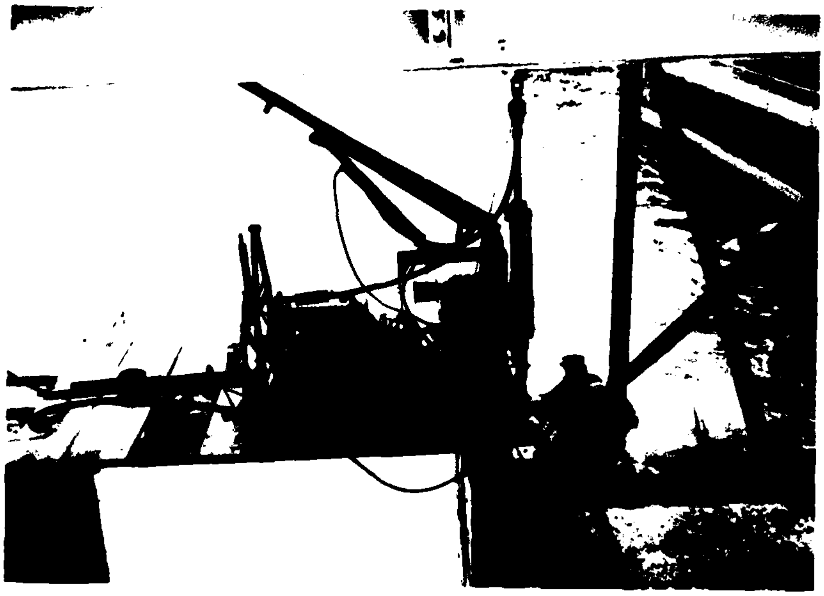
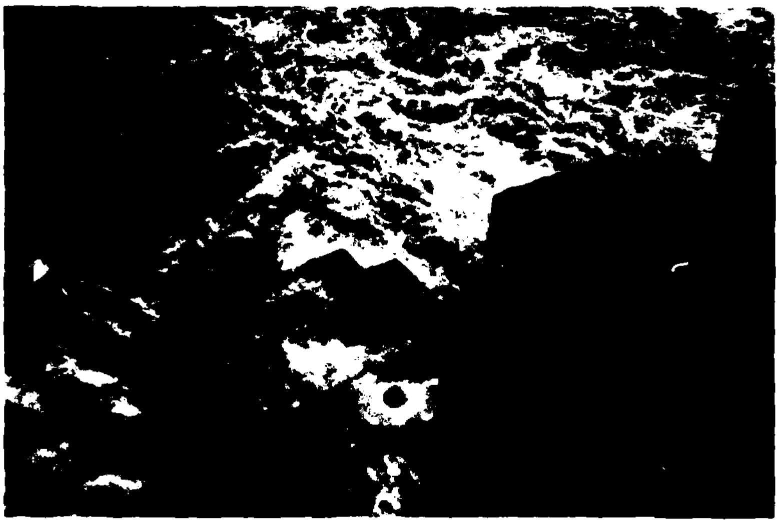


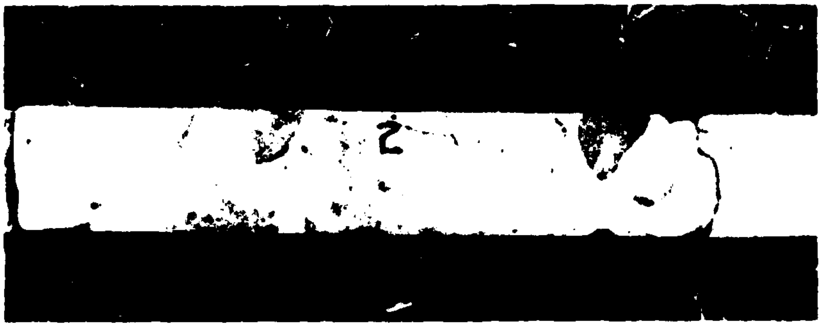
Figure 8. Platform and drilling rig coring pier 1, Winnibigoshish Dam

a. Pier 1



b. Pier 3

Figure 9. Capped core holes



a. Core depth 2 ft

b. Core depth 6 ft

c. Supporting timber from under dam, core depth 13 ft

Figure 10. Representative core pictures, Pier 1, core W-P1

Figure 11. Core and cut sections

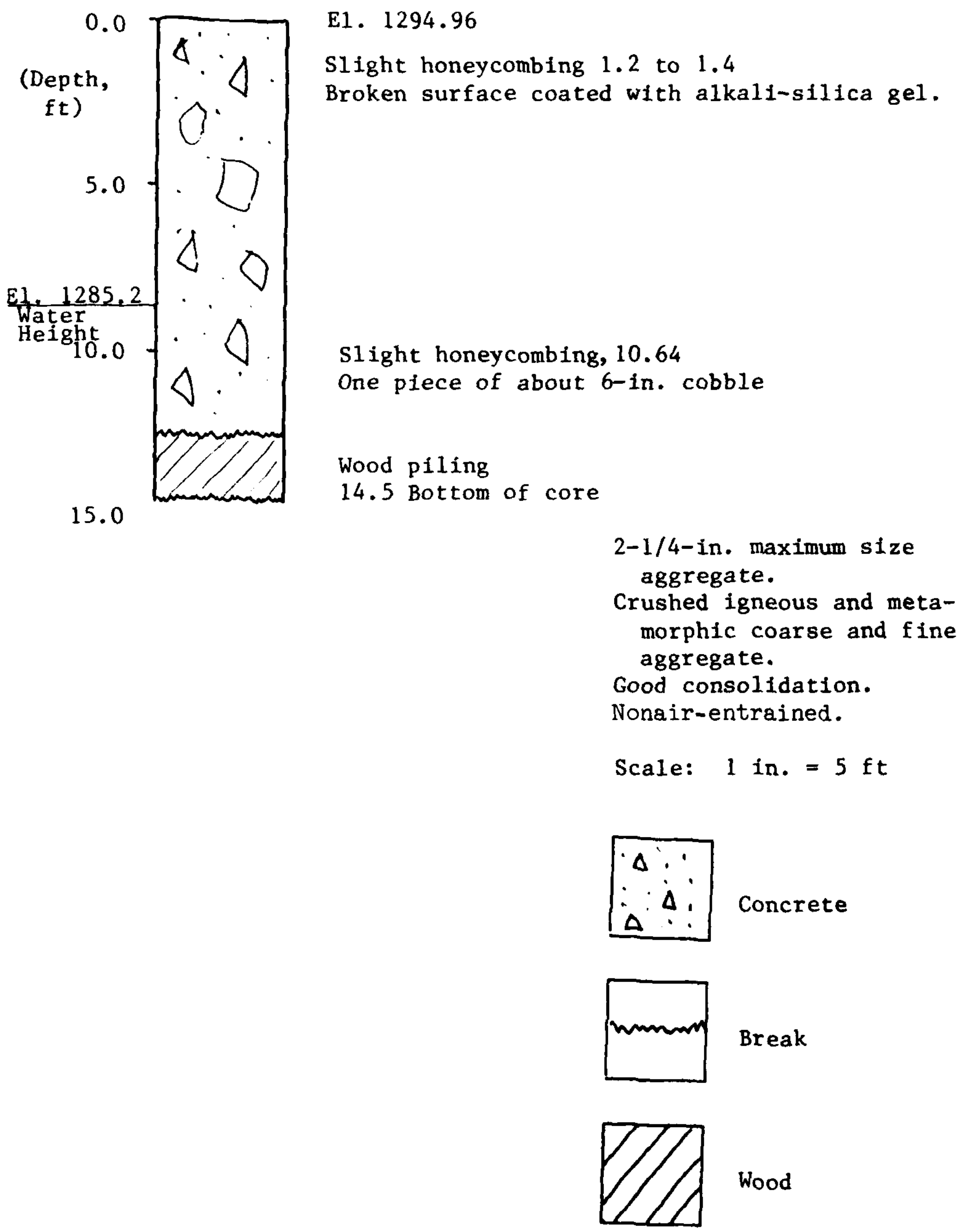
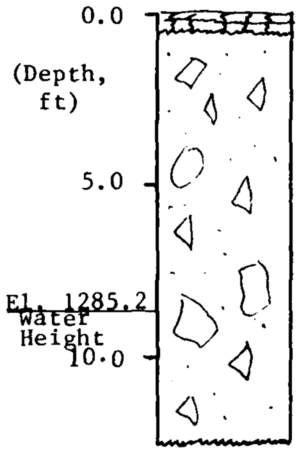


Figure 12. Vertical NX concrete core, W-P1, Winnibigoshish Dam



El. 1294.96
Fragmented to 0.7
Slight honeycombing, 1.2 to 1.45
Fragmented 2.15 to 2.34

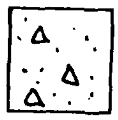
Breaks every 0.1 to 0.2 ft

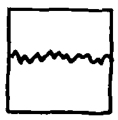
Slightly honeycombed, 8.0
Slightly honeycombed, 9.2
Slightly honeycombed, 10.4 to 10.9

End of core, 12.4

2-in. maximum size aggregate.
Crushed igneous and metamorphic coarse and fine aggregate.
Good consolidation.
Nonair-entrained.

Scale: 1 in. = 5 ft


Concrete


Break

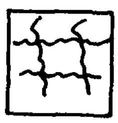
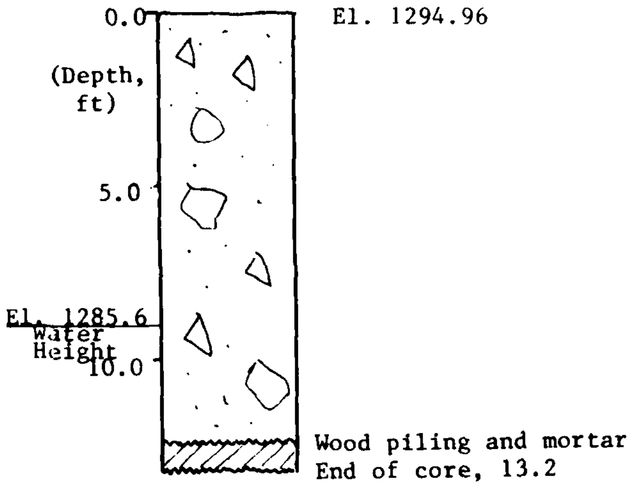

Fragmented

Figure 13. Vertical NX concrete core, W-P1A, Winnibigoshish Dam



1-1/2-in. maximum size aggregate.
Crushed igneous and metamorphic coarse and fine aggregate.
Good consolidation.
Nonair-entrained.

Scale: 1 in. = 5 ft

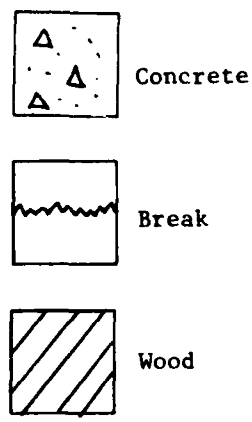
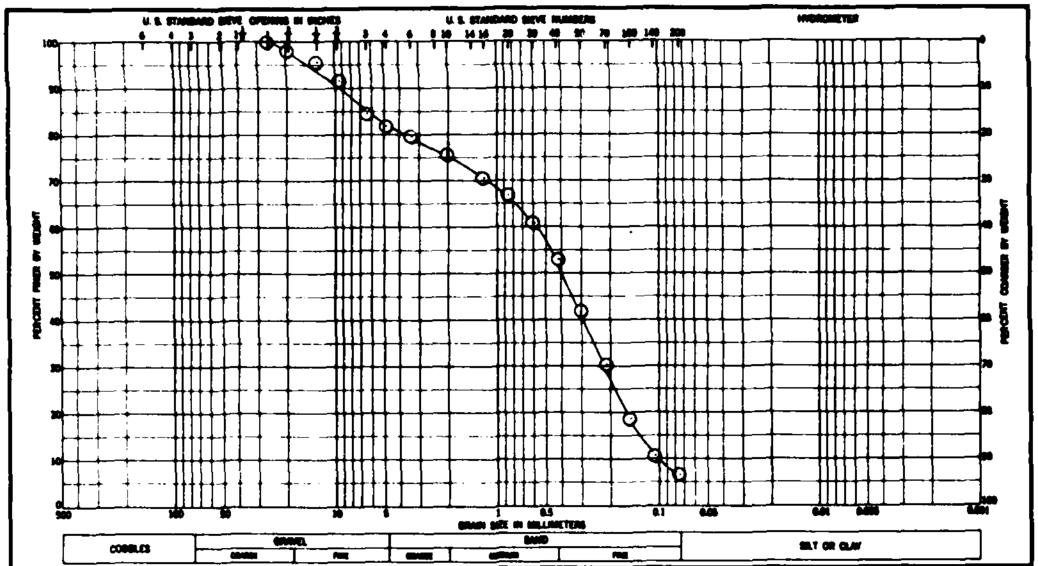


Figure 14. Vertical NX concrete core, W-P3, Winnibigoshish Dam



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI		
		GRAVELLY SILTY SAND (SP-SM)					Project	STABILITY ANALYSIS
		BROWN						
							Area	WINNIBIGOSHISH DAM
		INSUFFICIENT SAMPLE FOR					Boring No.	W-P1A
		ACCURATE GRADATION						
GRADATION CURVES							Date	19 MAR 80

Figure 15. Foundation soil sieve analysis and classification, sample 1, hole W-P1A

Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI		
		CLAYEY SAND (SC) WITH		23	11	12	Project	STABILITY ANALYSIS
		GRAVEL GRAY						
							Area	WINNIBIGOSHISH DAM
		INSUFFICIENT SAMPLE FOR					Boring No.	W-P1A
		ACCURATE GRADATION						
GRADATION CURVES							Date	20 MAR 80

Figure 16. Foundation soil sieve analysis and classification, sample 2, hole W-P1A

LIQUID AND PLASTIC LIMIT TESTS

For use of this form, see EM 1110-2-1906.

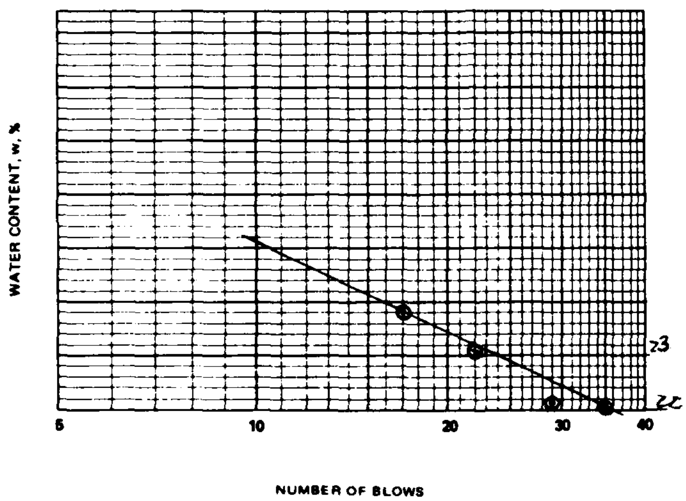
DATE 2. 19-80

PROJECT Stability Analysis

BORING NO. W-P1A SAMPLE NO. 14.6-16.1

LIQUID LIMIT

	RUN NO.		1	2	3	4	5	6
	TARE NO.		168A	167A	51A	110A		
WEIGHT IN GRAMS	TARE PLUS WET SOIL		29.01	28.14	21.99	21.39		
	TARE PLUS DRY SOIL		27.03	26.37	19.99	19.52		
	WATER	W_w	1.98	1.77	2.00	1.87		
	TARE		18.72	18.71	10.95	11.01		
	DRY SOIL	W_s	8.31	7.66	9.04	8.51		
	WATER CONTENT, %	w	23.8	23.1	22.1	22.0		
	NUMBER OF BLOWS		17	22	29	35		



LL 23.0

PL 11.0

PI 12.0

Symbol from plasticity chart (CL)

22.9
10.9
12.9

PLASTIC LIMIT

	RUN NO.		1	2	3	4	5	NATURAL WATER CONTENT
	TARE NO.		34A	72A				
WEIGHT IN GRAMS	TARE PLUS WET SOIL		18.05	19.06				
	TARE PLUS DRY SOIL		17.34	18.28				
	WATER	W_w	0.71	0.78				
	TARE		10.87	11.00				
	DRY SOIL	W_s	6.47	7.28				
	WATER CONTENT, %	w	11.0	10.7				
	PLASTIC LIMIT			10.9				

REMARKS SAVE ALL MAT'L FOR SIEVE [illegible]
[illegible] (CL)

TECHNICIAN E.O COMPUTED BY E.O CHECKED BY

Figure 17. Liquid and plastic limit, hole W-P1A

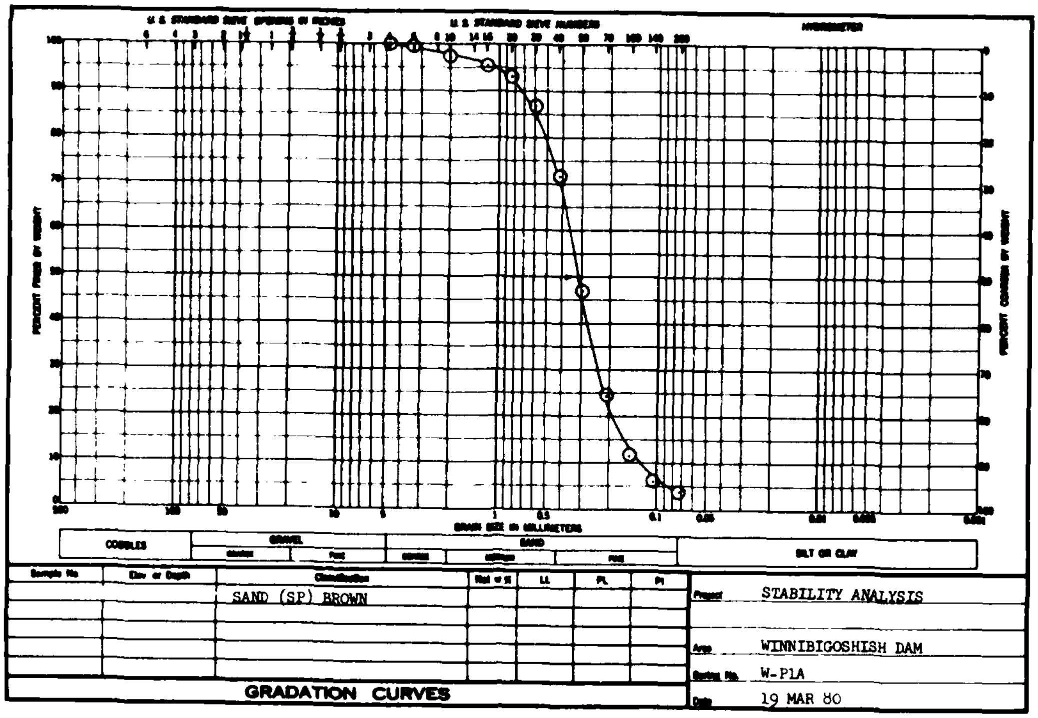


Figure 18. Foundation soil sieve analysis and classification, sample 3, hole W-P1A

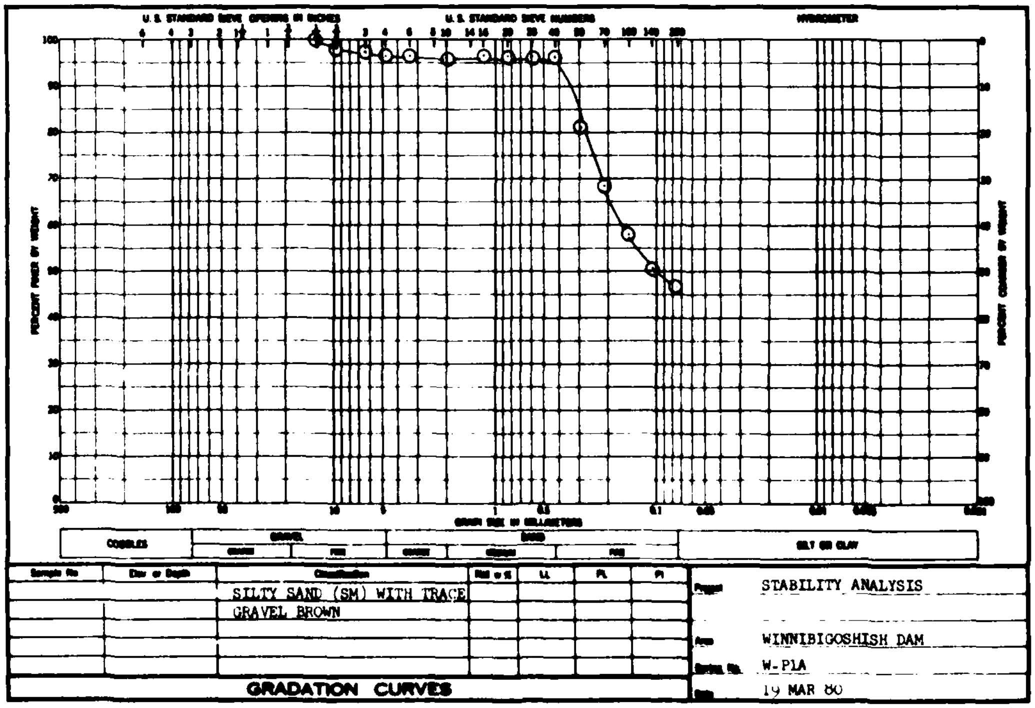


Figure 19. Foundation soil sieve analysis and classification, sample 4, hole W-P1A

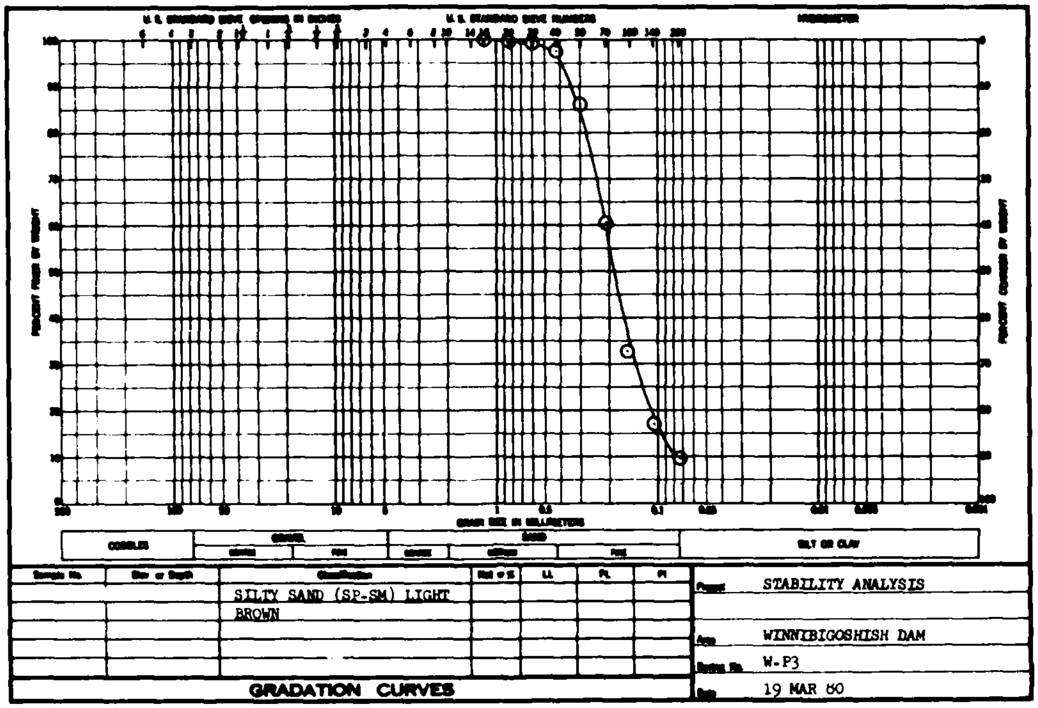


Figure 20. Foundation soil sieve analysis and classification, sample 1, hole W-P3

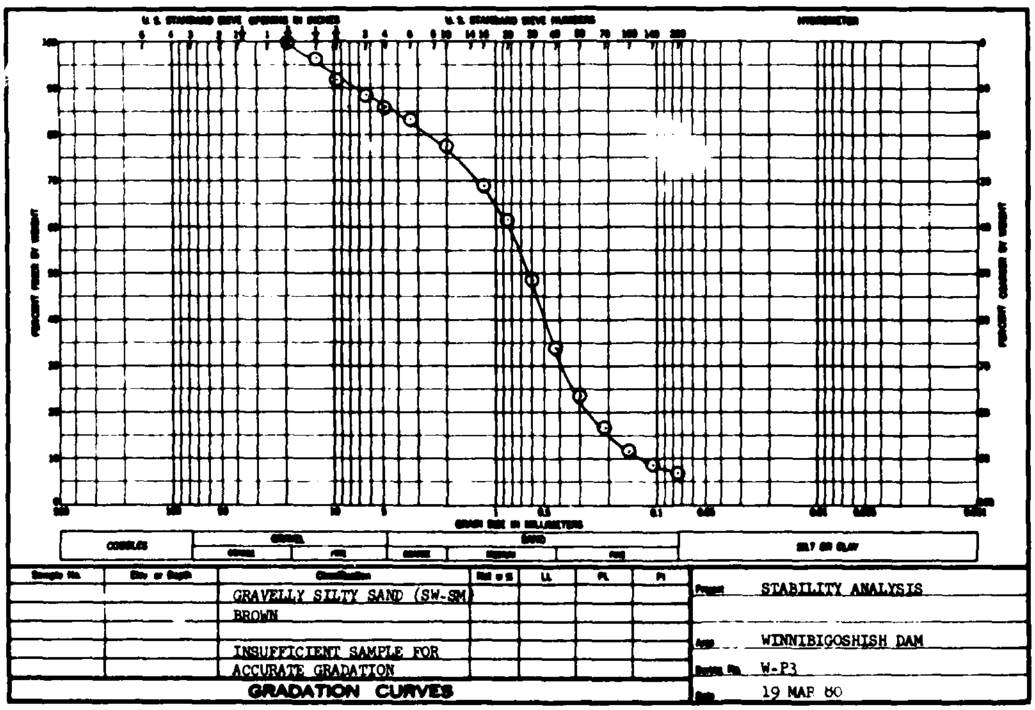
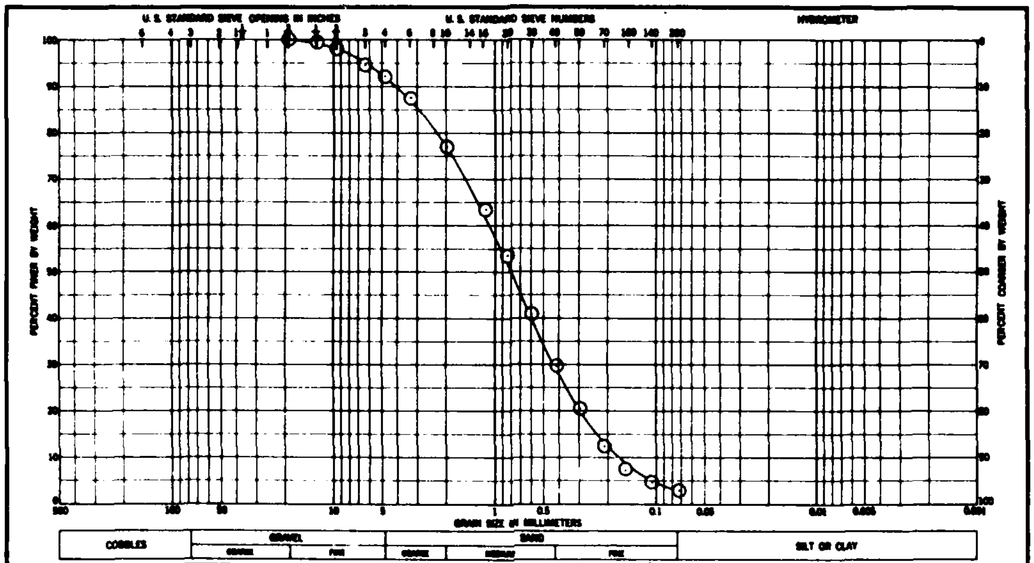


Figure 21. Foundation soil sieve analysis and classification, sample 2, hole W-P3

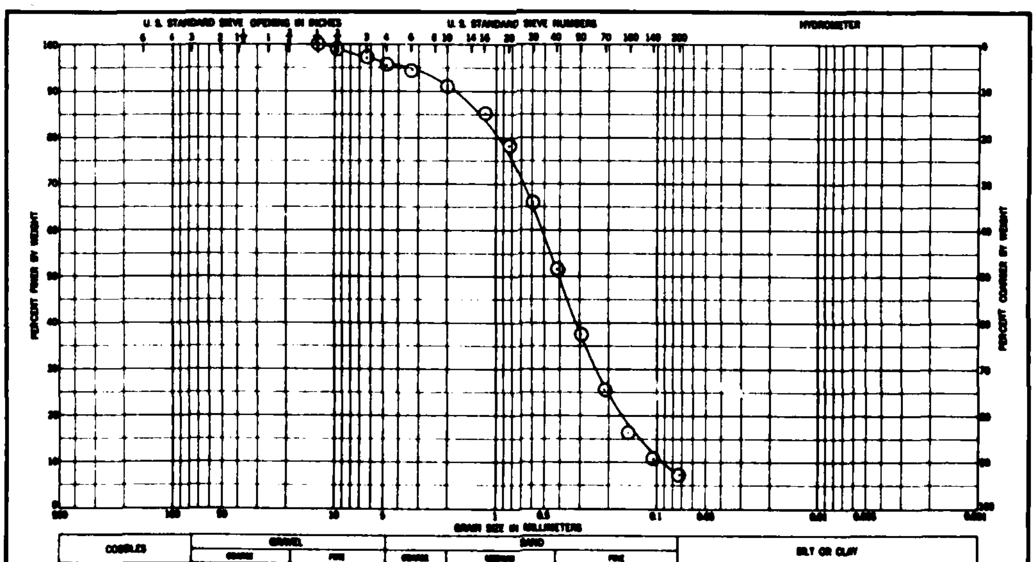


Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
		SAND (SP) WITH GRAVEL BROWN				
		INSUFFICIENT SAMPLE FOR ACCURATE GRADATION				

Project: STABILITY ANALYSIS
Area: WINNIBIGOSHISH DAM
Boring No.: W-P3
Date: 19 MAR 80

GRADATION CURVES

Figure 22. Foundation soil sieve analysis and classification, sample 3, hole W-P3



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
		SILTY SAND (SP-SM) WITH TRACE GRAVEL BROWN				

Project: STABILITY ANALYSIS
Area: WINNIBIGOSHISH DAM
Boring No.: W-P3
Date: 19 MAR 80

GRADATION CURVES

Figure 23. Foundation soil sieve analysis and classification, sample 4, hole W-P3

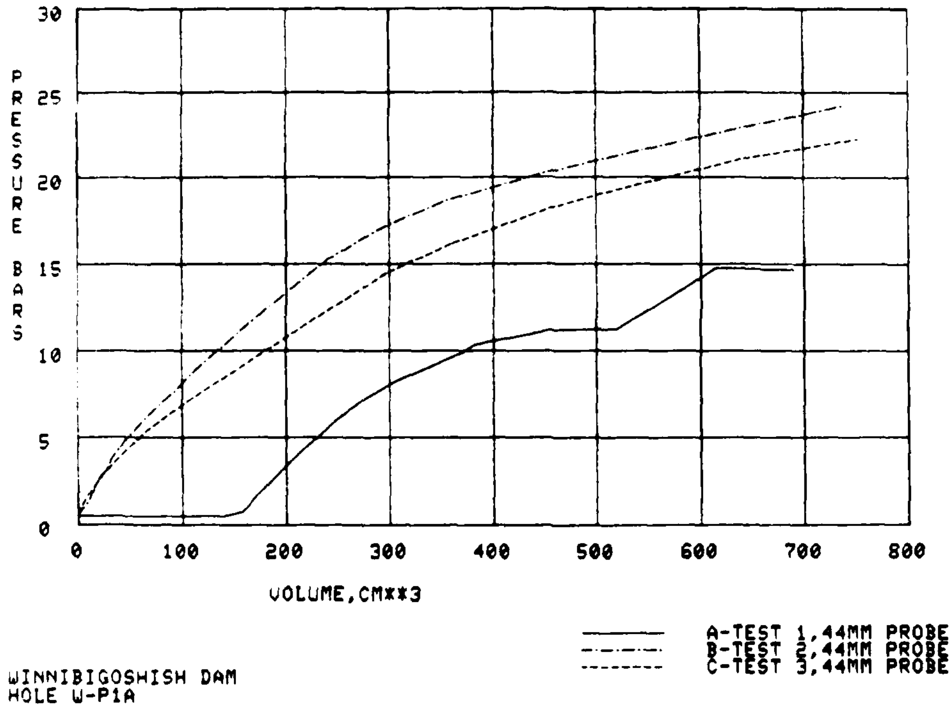


Figure 24. Pressure versus volume (metric units), W-PIA

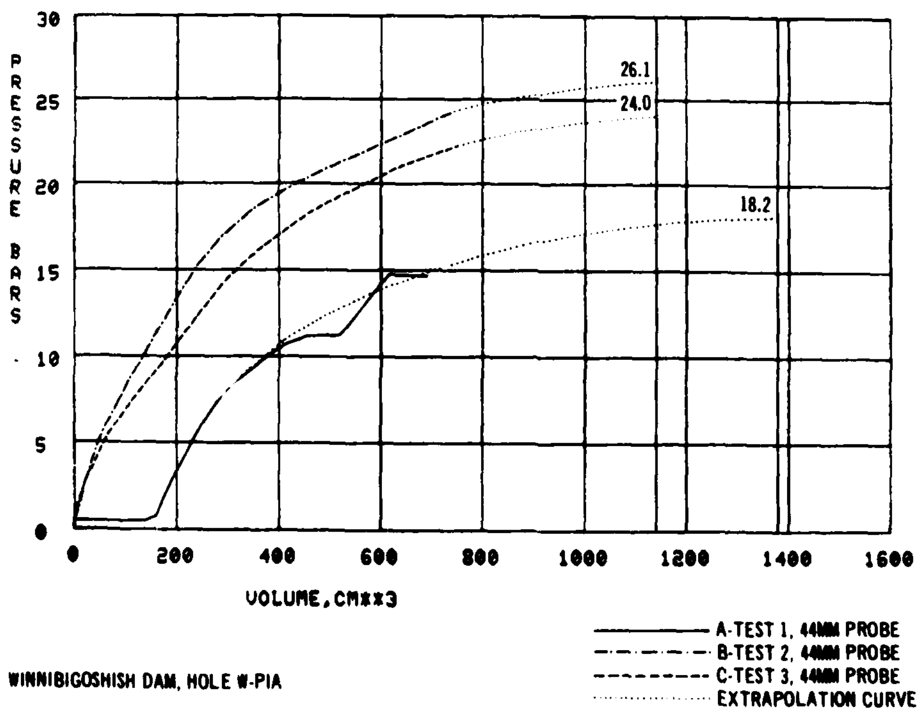


Figure 25. Pressure versus volume, curves extrapolated to obtain limit pressures, W-PIA

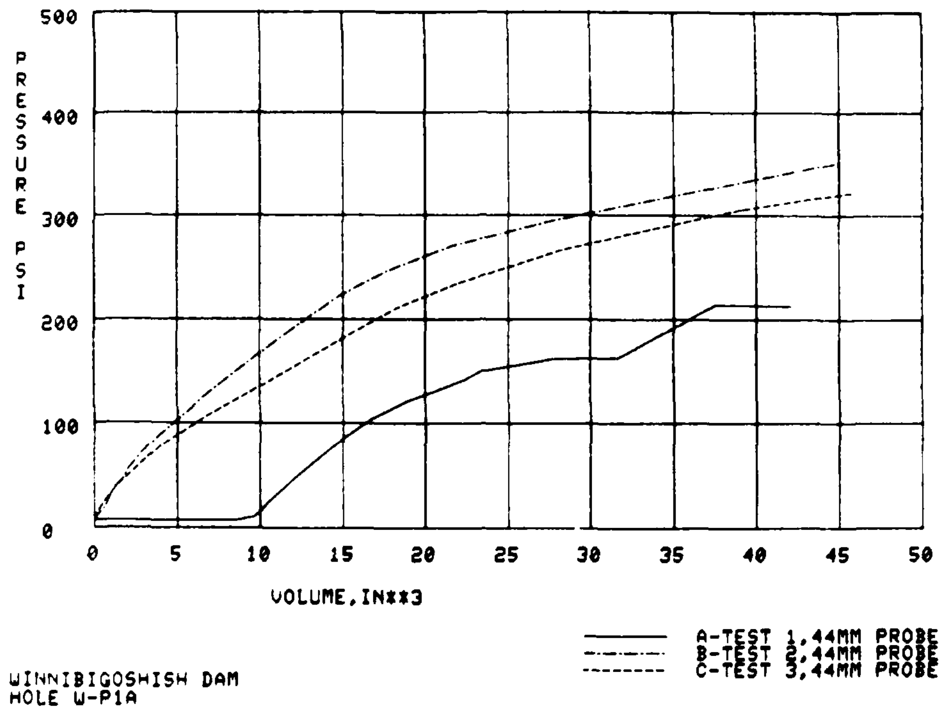


Figure 26. Pressure versus volume, W-P1A

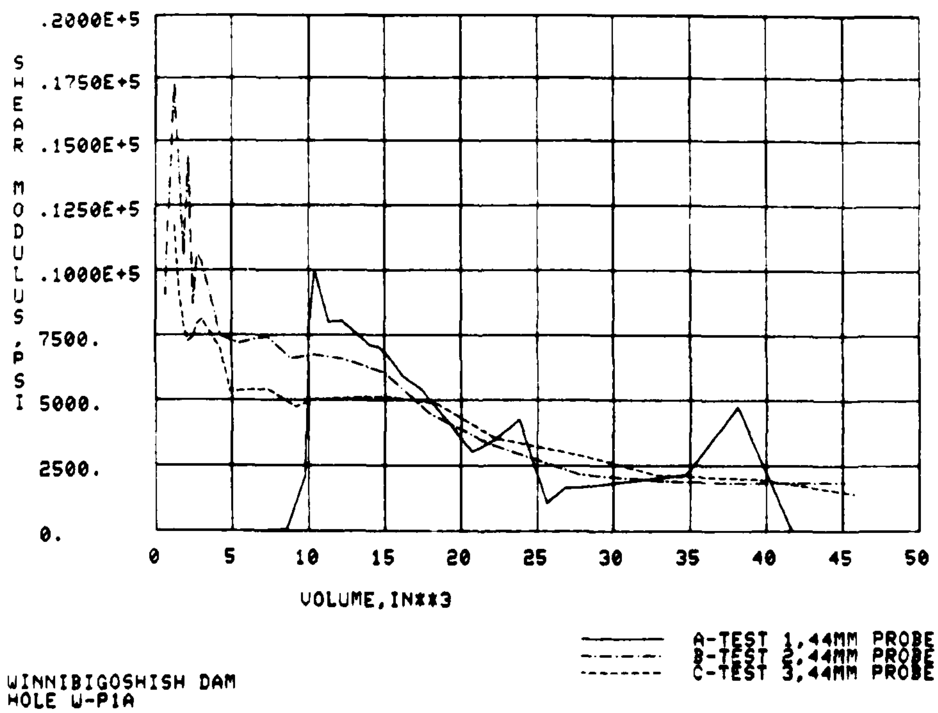


Figure 27. Shear modulus, W-P1A

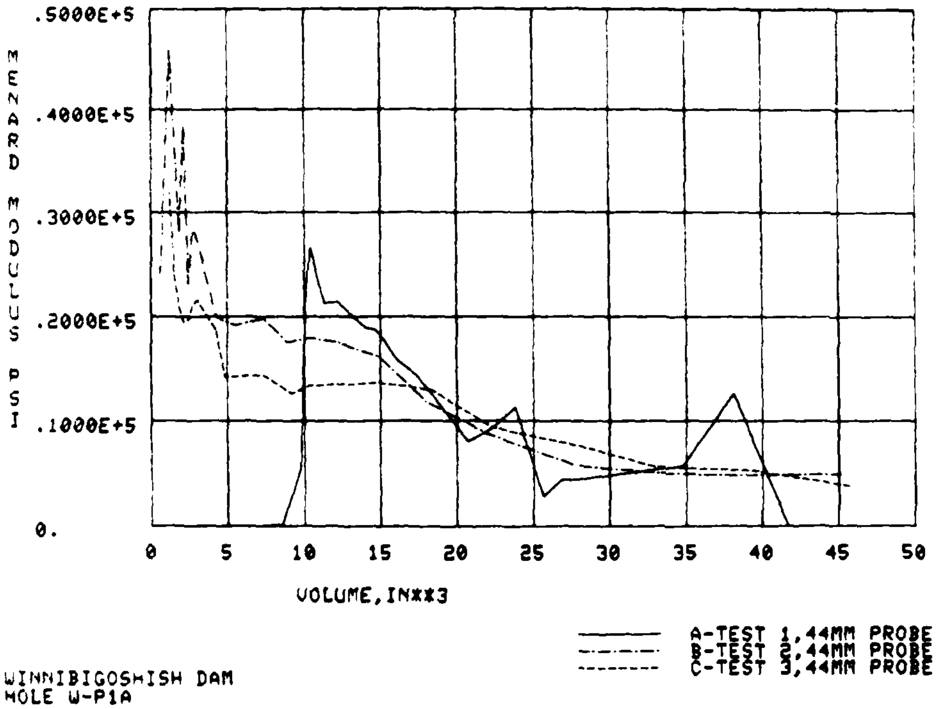


Figure 28. Ménard modulus, W-P1A

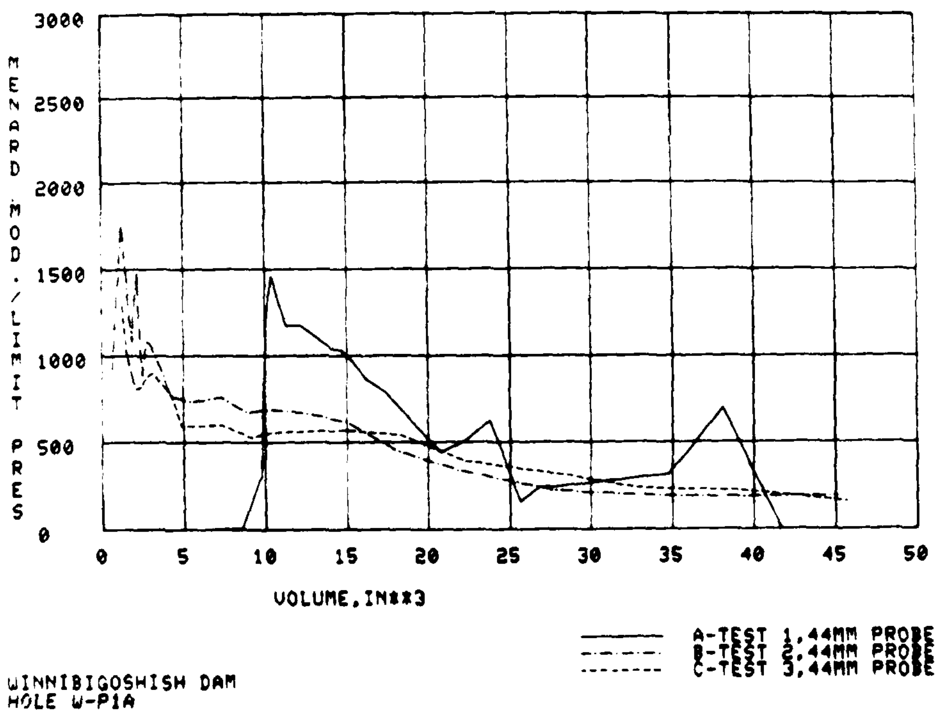


Figure 29. Ménard modulus divided by limit pressure, W-P1A

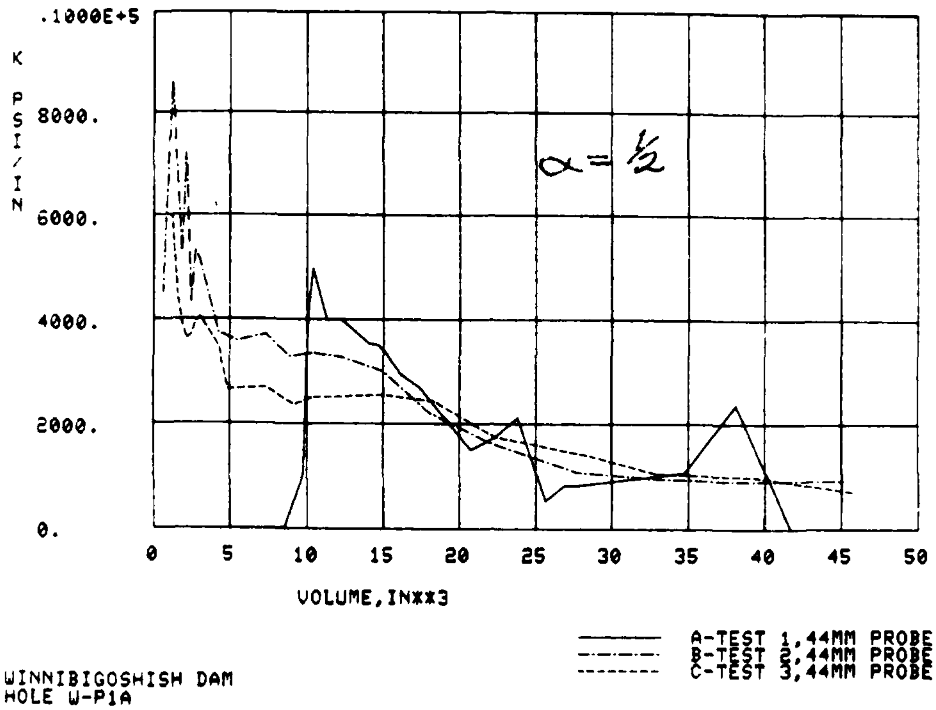


Figure 30. Modulus of subgrade reaction, W-P1A

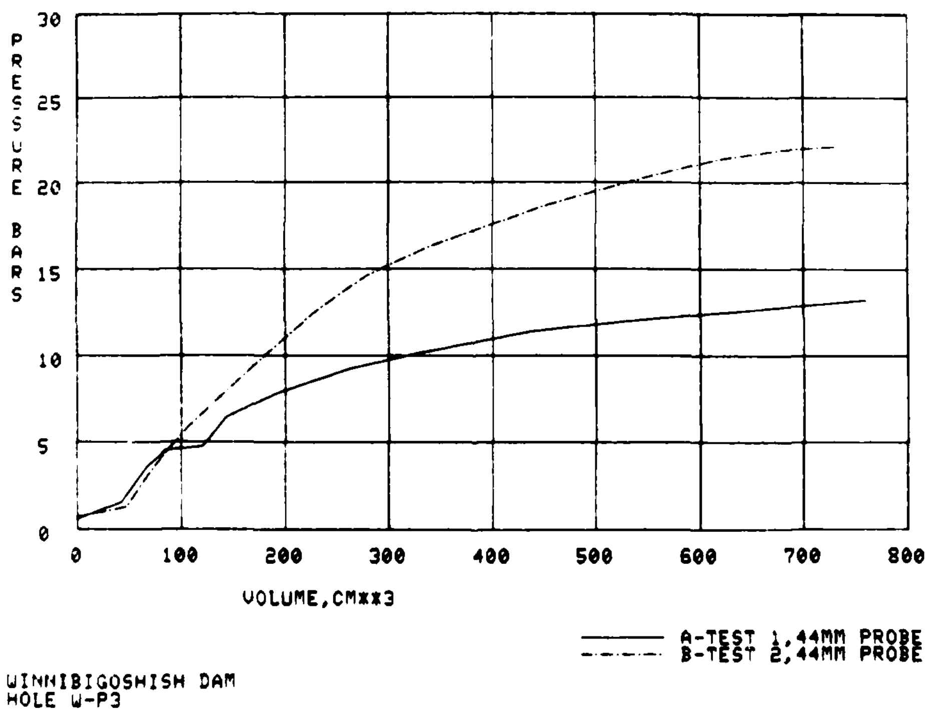


Figure 31. Pressure versus volume (metric units), W-P3

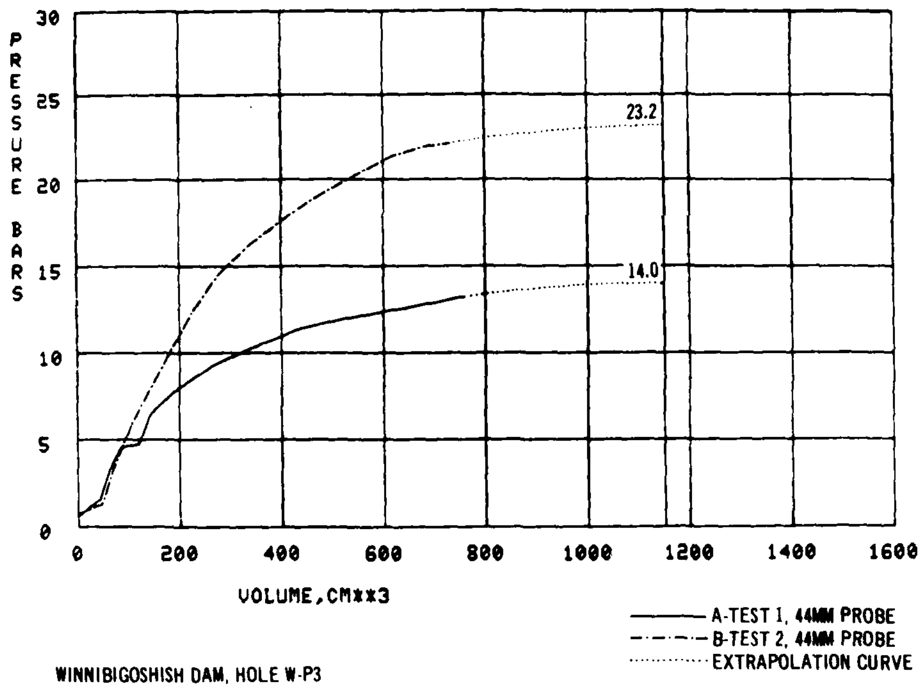


Figure 32. Pressure versus volume, curves extrapolated to obtain limit pressures, W-P1A

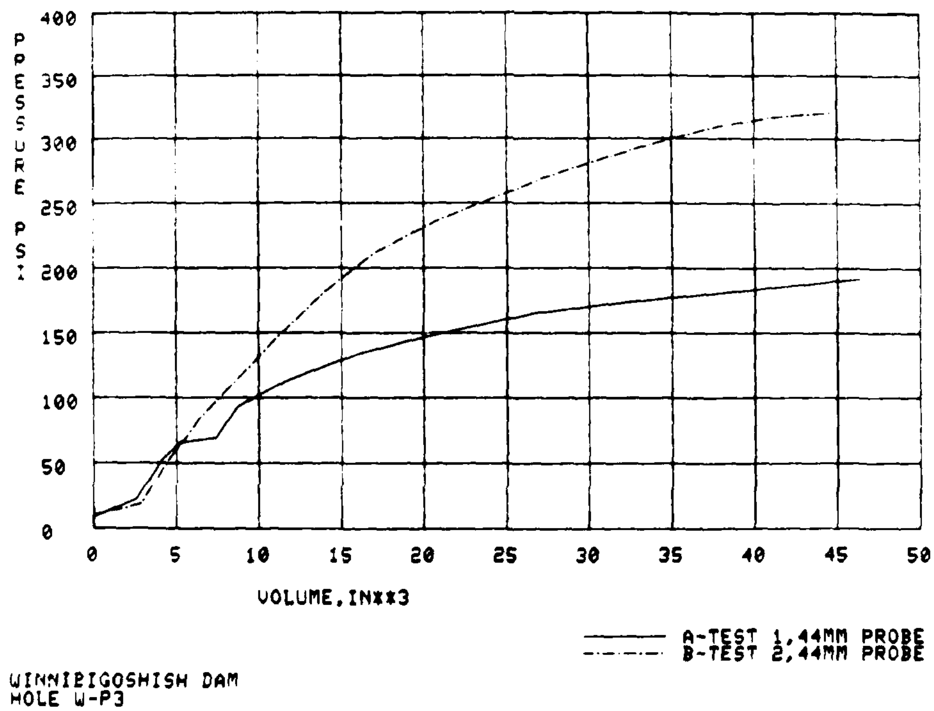


Figure 33. Pressure versus volume, W-P3

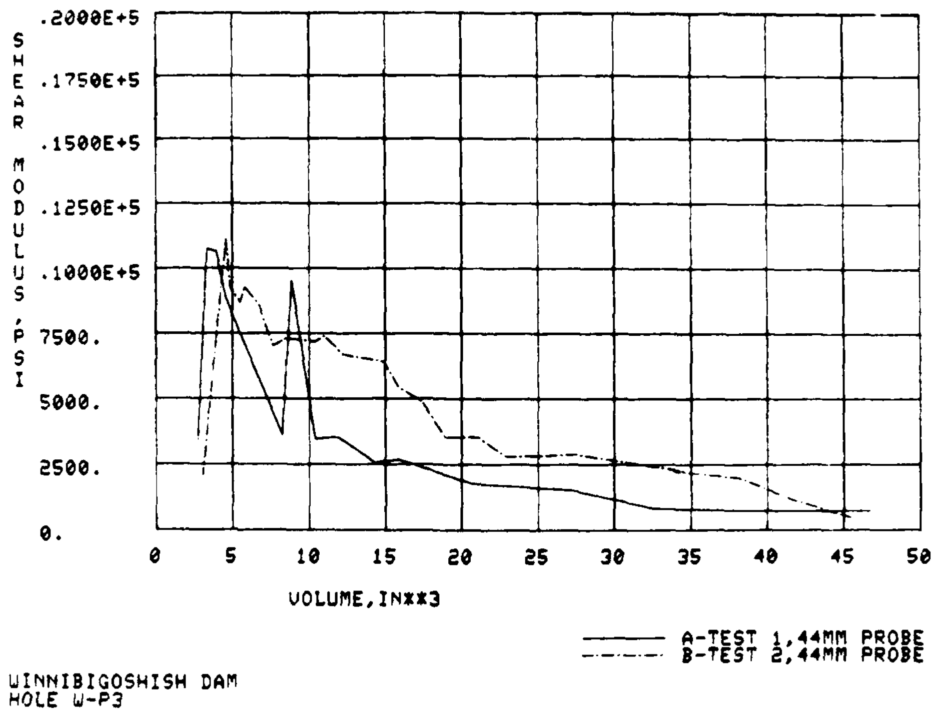


Figure 34. Shear modulus, W-P3

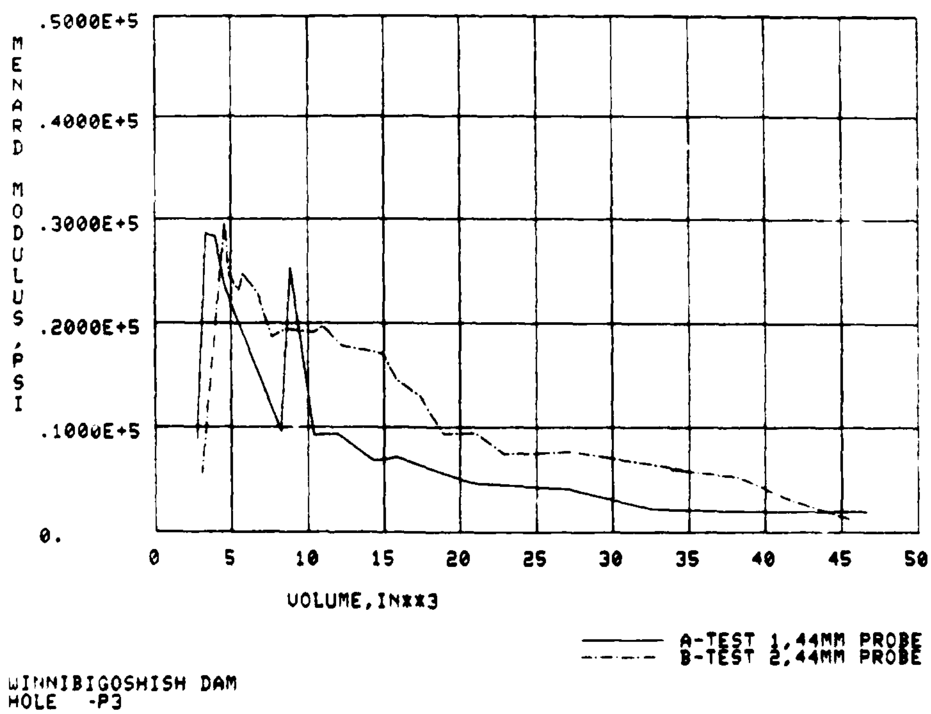


Figure 35. Ménard modulus, W-P3

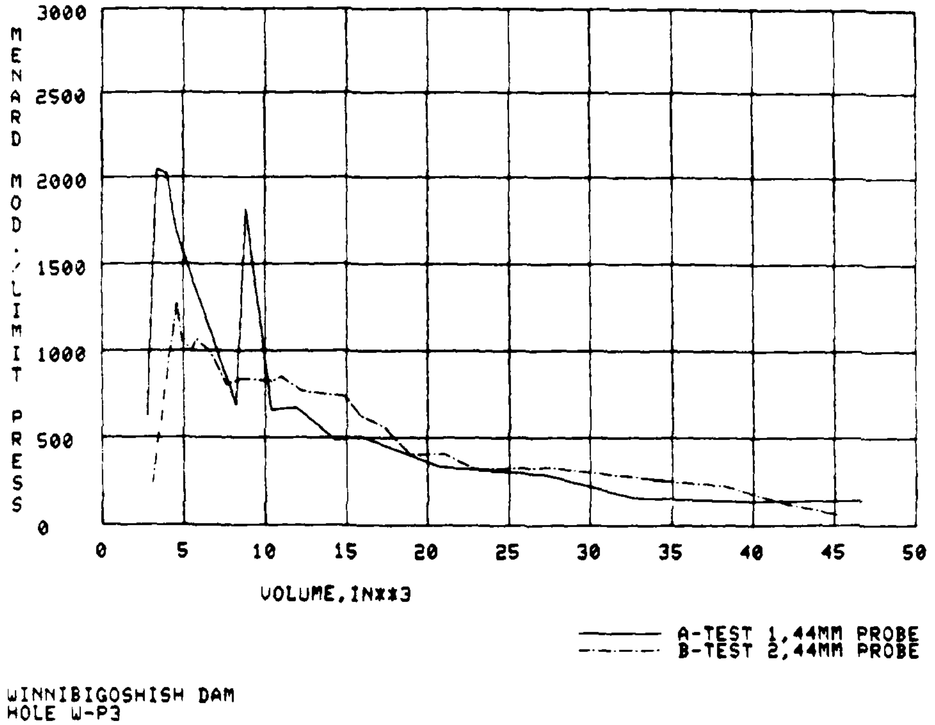


Figure 36. Ménard modulus divided by limit pressure, W-P3

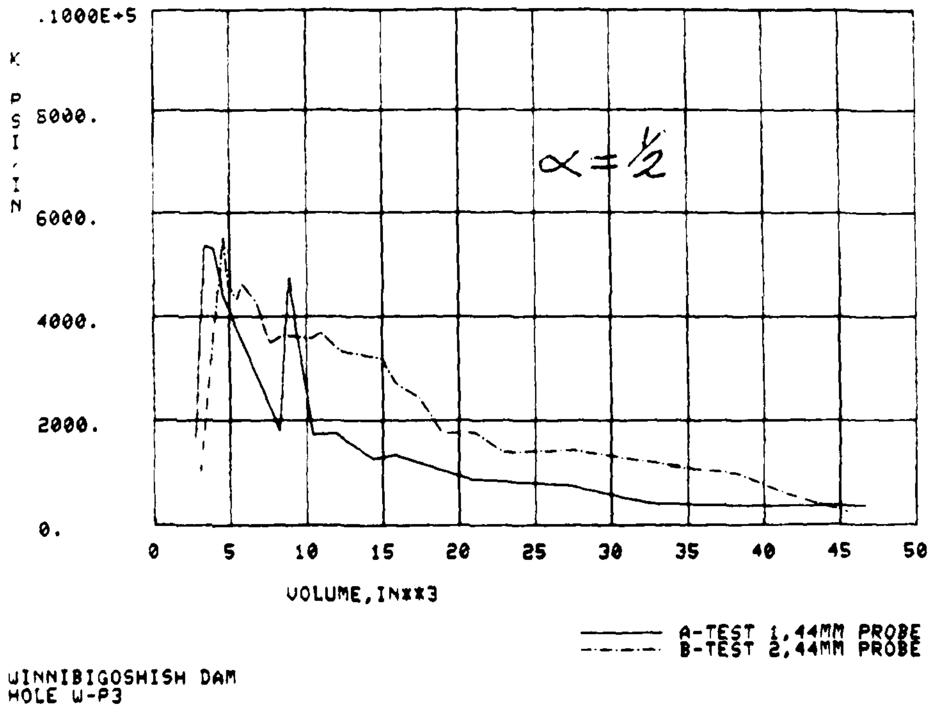


Figure 37. Modulus of subgrade reaction, W-P3

a. Concrete surface deterioration, abutment

b. Concrete surface deterioration, pier

Figure 38. Abutment and pier deterioration, Winnibigoshish Dam

a. Side of pier

b. Close view of concrete surface deterioration

Figure 39. Pier deterioration, Winnibigoshish Dam

Factor	F_V kips	F_H kips	Arm_x ft	Arm_y ft	M_z ft-K
W_{conc} = (0.15)[25.0 x 11.0 x (1294.96 - 1282.64)]	+508.20		+12.50		+6,352.50
(0.15)[12.0 x 11.0 x (1304.36 - 1282.64)]	+430.06		+31.00		+13,331.86
(0.15)(2)[(2.0)(2.5)(1303.86 - 1294.96)]	+13.35		+24.00		+320.40
(0.15)(2)[2.0(2.5)(1302.86 - 1294.96)]	+11.85		+22.00		+260.70
(0.15)(2)[(2.0)(2.5)(1301.86 - 1294.96)]	+10.35		+20.00		+207.00
(0.15)(2)[(2.0)(2.5)(1300.86 - 1294.96)]	+8.85		+18.00		+159.30
(0.15)(2)[(2.0)(2.5)(1299.86 - 1294.96)]	+7.35		+16.00		+117.60
(0.15)(2)[(2.0)(2.5)(1298.86 - 1294.96)]	+5.85		+14.00		+81.90
(0.15)(2)[(2.0)(2.5)(1297.86 - 1294.96)]	+4.35		+12.00		+52.20
(0.15)(2)[(2.0)(2.5)(1296.86 - 1294.96)]	+2.85		+10.00		+28.50
(0.15)(2)[2.0(2.5)(1295.86 - 1294.96)]	+1.35		+8.00		+10.80
	+1004.41				+20,922.76
W_{Bridge} Superstructure (10)(25.0)(0.045)	+11.25		+11.25		+126.56
Assume W 10 x 45 for stringers and floor beams					
(1)(24.0)(0.045) (Floor beam)	+1.08		+11.25		+12.15
(2)(14.5)(0.020)	+0.59		+11.25		+6.64
Assume W 8 x 20 for columns					
(0.145)(0.5)(24.0)(25.0) Asphalt bridge deck γ compacted	+43.50		+11.25		+489.38
	56.42				634.73
$P_{Head\ Water}$ = $(0.0625)(1301.86 - 1282.64)^2(0.5)(25.0)$		-288.60		6.41	-1,849.93
$P_{Tail\ Water}$ = $(0.0625)(1283.86 - 1282.64)^2(0.6)(0.5)(25.0)$		0.70		0.41	0.29
Uplift = $(-11)(37)[(0.0625)[1283.86 - 1282.64] + \frac{(14)(18)(0.0625)}{14 + 37 + 14}]$	-129.65		18.50		-2,398.53
$-(1/2)(11)(37)[(0.0625)[1301.86 - 1283.86][\frac{37}{14 + 37 + 14}]]$	-130.32		24.67		-3,214.99
	-259.97				-5,613.52

Figure 40. Normal operation (continued)

Factor	F_V kips	F_H kips	Arm_x ft	Arm_y ft	M_z ft-K
$e = \frac{14,094.33}{800.86} = 17.60$ ft					
Total	800.86	-287.90			14,094.33

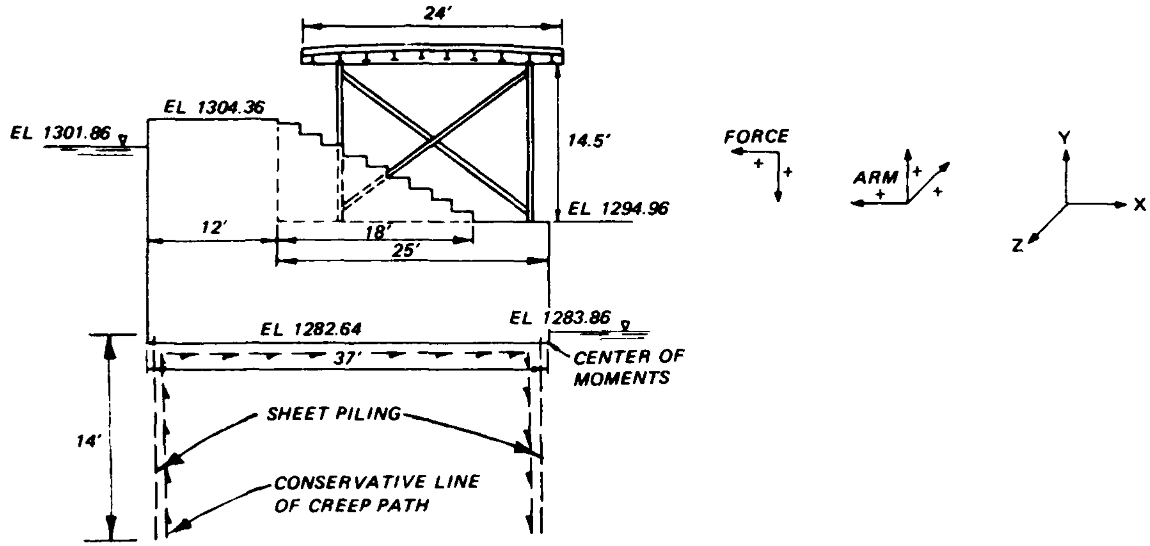


Figure 40. (concluded)

Factor	F_V kips	F_H kips	Arm_x ft	Arm_y ft	M_z ft-K
Normal operation loadings	800.86	-287.90			14,094.33
Truck load (H15-44)	26.64		4.25		113.22
	827.5	-287.90			14,207.55

$$e = \frac{14,207.55}{827.5} = 17.17 \text{ ft}$$

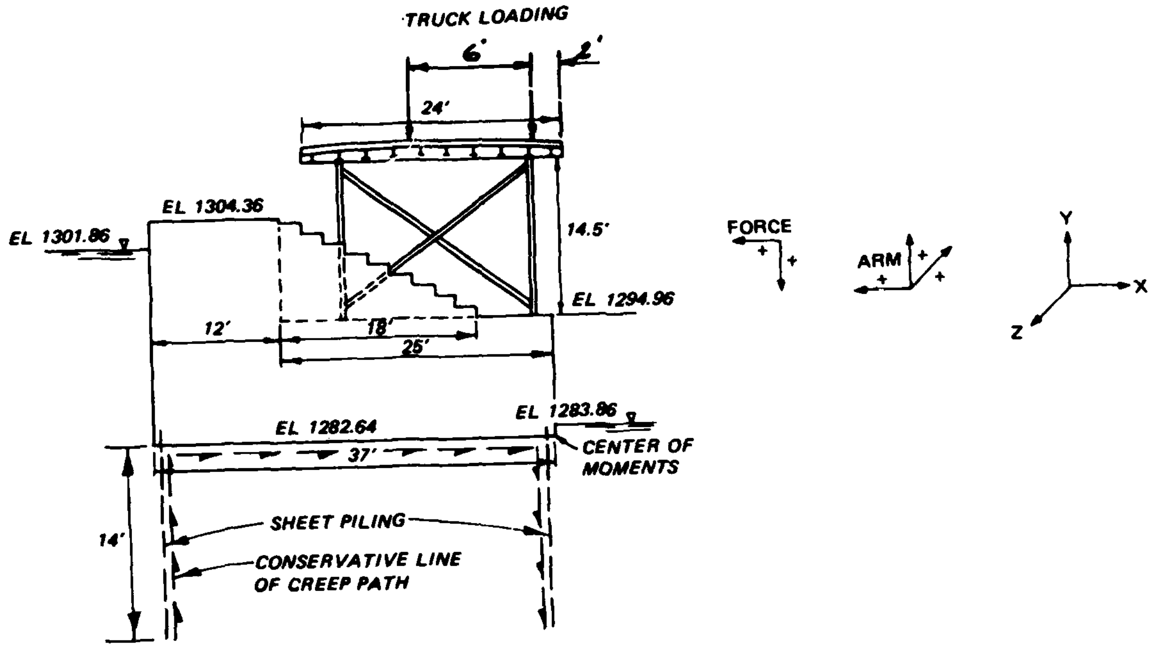


Figure 41. Normal operation with truck loading (H15-44)

Factor	F_V kips	F_H kips	Arm_x ft	Arm_y ft	M_z ft-K
Normal operation loadings	800.86	-287.90			14,094.33
Earthquake: (EM 1110-2-2200)					
$Pe_1 = (0.025)(1004.41 + 11.25 + 1.0^{-} + 0.59 + 43.50)$		-26.52		9.80	-259.90
$Pe_2 = (2/3)(51)(0.025)(19.22)^2(25)(1/1000)$		-7.85		7.69	-60.37
$e = \frac{13,774.06}{800.86} = 17.20$					
Total	800.86	-322.27			13,774.06

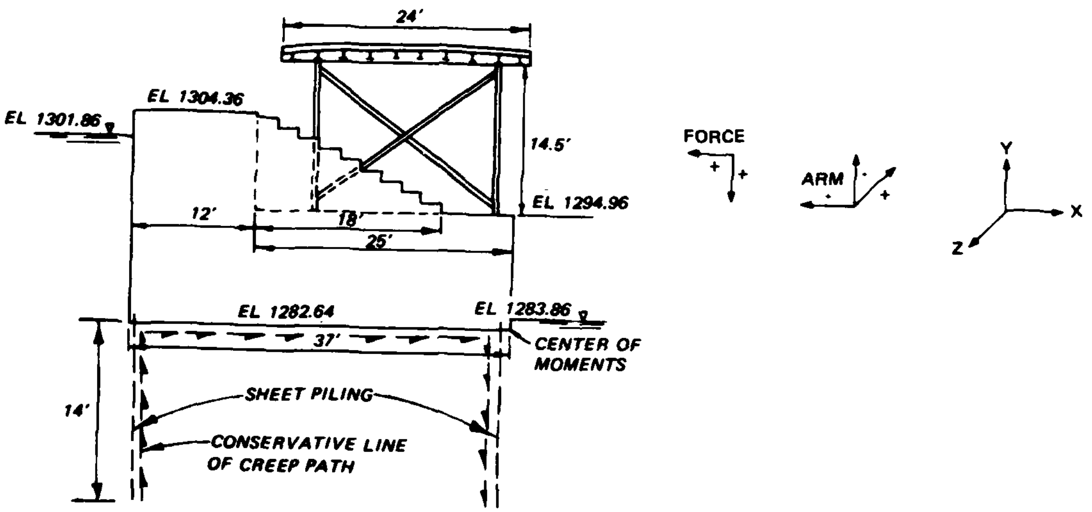


Figure 42. Normal operation with earthquake

Pier

Concrete

(508.20)(1/2)(1294.96 - 1282.64)	=	3130.51
(430.06)(1/2)(1304.36 - 1282.64)	=	4670.45
(13.35)[(1/2)(1303.86-1294.96) + (1294.96-1282.64)]	=	223.88
(11.85)[(1/2)(1302.86-1294.96) + (1294.96-1282.64)]	=	192.80
(10.35)[(1/2)(1301.86-1294.96) + (1294.96-1282.64)]	=	163.22
(8.85)[(1/2)(1300.86-1294.96) + (1294.96-1282.64)]	=	135.14
(7.35)[(1/2)(1299.86-1294.96) + (1294.96-1282.64)]	=	108.56
(5.85)[(1/2)(1298.86-1294.96) + (1294.96-1282.64)]	=	83.48
(4.35)[(1/2)(1297.86-1294.96) + (1294.96-1282.64)]	=	59.90
(2.85)[(1/2)(1296.86-1294.96) + (1294.96-1282.64)]	=	37.82
(1.35)[(1/2)(1295.86-1294.96) + (1294.96-1282.64)]	=	17.24
		8823.00

$$Arm_y = \frac{8823.00}{1004.41} = 8.78 \text{ ft}$$

Bridge

Steel

(11.25 + 1.08)(1294.96 - 1282.64 + 15)	=	336.86
(0.59)(1294.96 - 1282.64 + (1/2)(14.5))	=	11.55
		348.41

$$Arm_y = \frac{348.41}{11.25 + 1.08 + 0.59} = 26.97 \text{ ft}$$

Asphalt

(43.50)(1294.96 - 1282.64 + 15.75) = 1221.05

$$Arm_y = 1294.96 - 1282.64 + 15.75 = 28.07 \text{ ft}$$

Pier and Bridge

$$Arm_y = \frac{8823 + 348.41 + 1221.05}{1004.41 + 11.25 + 1.08 + 0.59 + 43.50} = 9.80 \text{ ft}$$

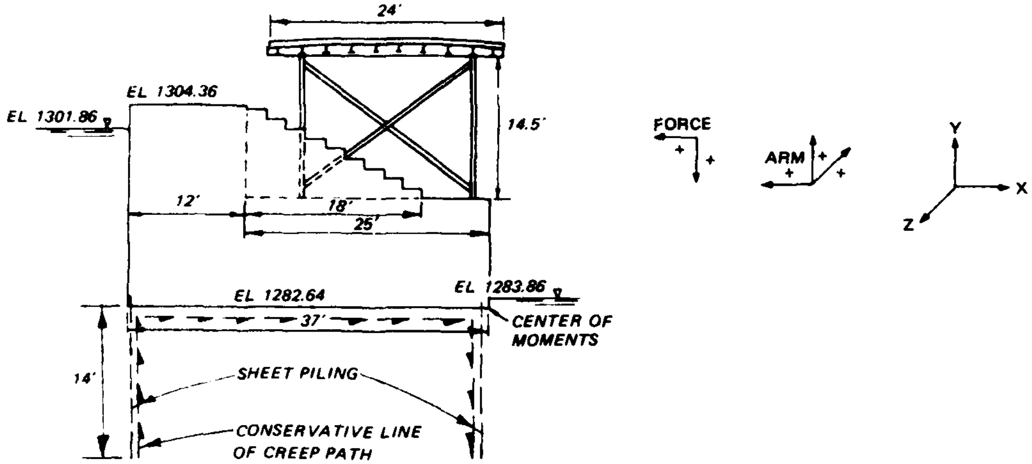


Figure 43. Normal operation with earthquake, centroid weights in YZ plane

Factor	F_V kips	F_H kips	Arm_x ft	Arm_y ft	M_z ft-K
Normal operation loading	800.86	-287.90			14,094.33
Ice load = (1)(5)(25)		-125.00		18.72	-2,340.00
$e = \frac{11,754.33}{800.86} = 14.68$ ft					
Total	800.86	-412.90			11,754.33

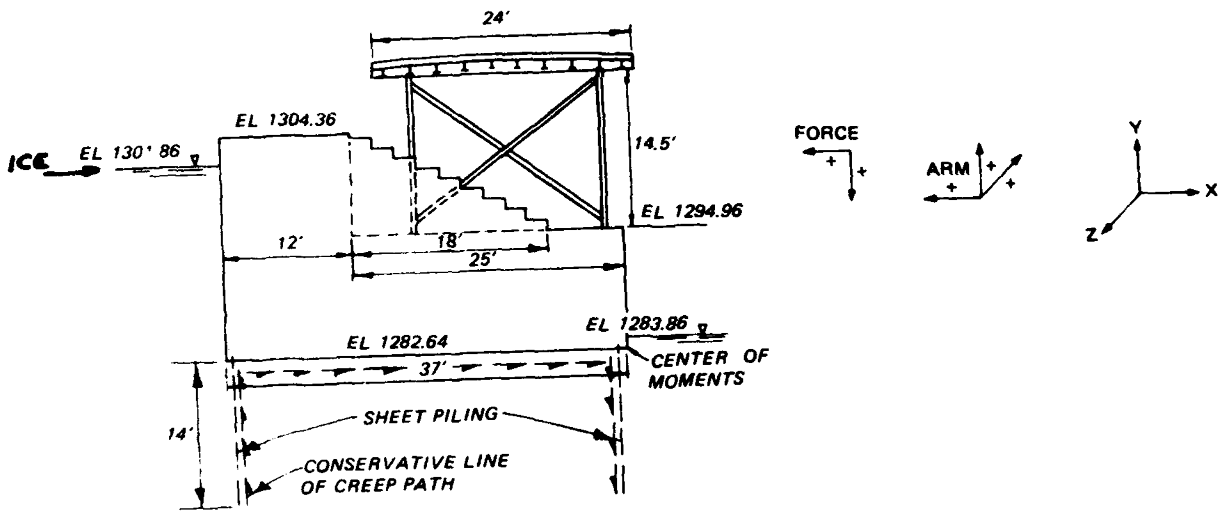


Figure 44. Normal operation with ice

Factor	F_V kips	F_H kips	Arm_x ft	Arm_y ft	M_z ft-K
W_{Conc} (See calculations for normal operation)	1004.41				20,922.76
W_{Bridge} (See calculations for normal operation)	56.42				634.73
$P_{Head\ Water} = (0.0625)(1304.36 - 1282.64)^2(0.5)(25)$		-368.56		7.24	-2,668.37
$P_{Tailwater} = (0.0625)(125.86 - 1282.64)^2(0.5)(0.6)(25)$		4.86		1.07	5.20
Uplift = $(-11)(37)[(0.0625)[1285.86-1282.64] + \frac{(14)(18.5)(0.0625)}{14 + 37 + 14}]$	-183.27		18.50		-3,390.50
$-(1/2)(11)(37)[(0.0625)[1304.36-1285.86][\frac{37}{14 + 37 + 14}]$	-133.94		24.67		-3,304.30
	-317.21				-6,694.80

$$e = \frac{12,194.32}{743.62} = 16.40 \text{ ft}$$

Total	743.62	-363.7			12,194.32

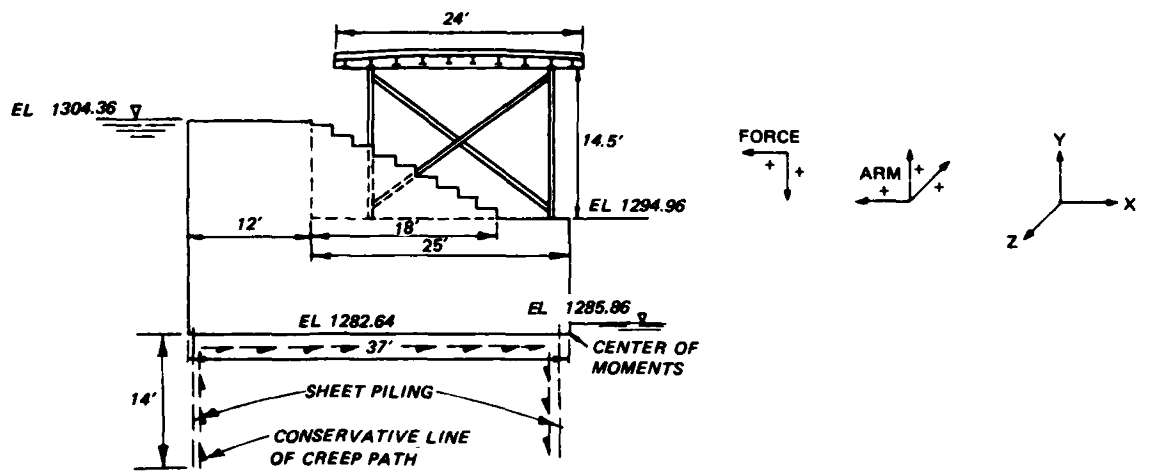
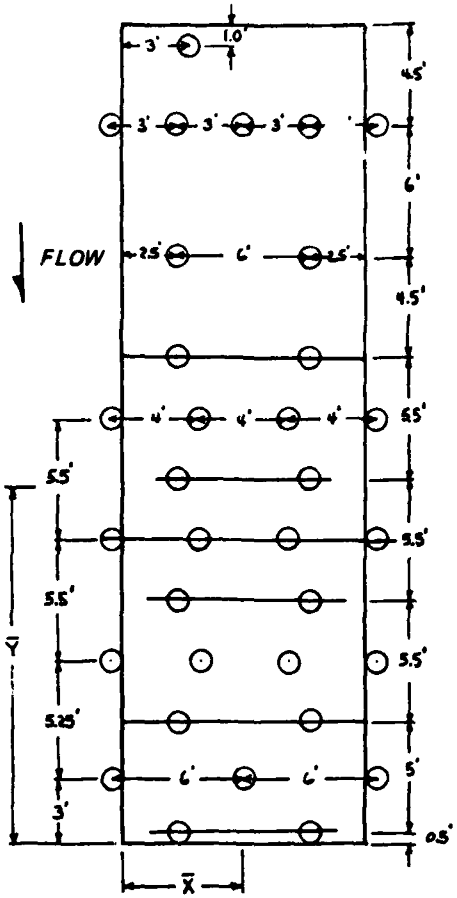


Figure 45. High-water condition

VERTICAL PILES



Location of Centroid of Pile Group

$$\bar{Y} = \frac{(1)(36) + (2)[0.5 + 5.5 + 11 + 16.5 + 22 + 26.5] + (3)(3) + (4)[8.25 + 13.75 + 19.25] + (5)(32.5)}{33}$$

$$\bar{Y} = 16.26 \text{ ft}$$

$$\bar{X} = \frac{(5)(11.5) + (7)(8.5) + (3)(7.5) + (2)(5.5) + (3)(3.5) + (1)(3) + (7)(2.5) + (5)(-0.5)}{33}$$

$$\bar{X} = 5.42 \text{ ft}$$

Moment of Inertia of Pile Group About Centroid of Pile Group

$$I = I_o + Ad^2$$

$$I_{XX} = \frac{(33)(\pi)(0.5)^4}{4} + [\pi(0.5)^2][(2)[(16.26 - 0.5)^2 + (16.26 - 5.5)^2 + (16.26 - 11)^2 + (16.26 - 16.5)^2 + (16.26 - 22)^2 + (16.26 - 26.5)^2] + (3)(16.26 - 3)^2 + (4)[(16.26 - 8.25)^2 + (16.26 - 13.75)^2 + (16.26 - 19.25)^2] + (5)(16.26 - 32.5)^2 + (1)(16.26 - 36)]$$

$$I_{XX} = 2839.1 \text{ ft}^4 = 5.8872 \times 10^7 \text{ in.}^4$$

$$I_{YY} = \frac{(33)(\pi)(0.5)^4}{4} + [\pi(0.5)^2][(5)(5.42 - 11.5)^2 + (7)(5.42 - 8.5)^2 + (3)(5.42 - 7.5)^2 + (2)(5.42 - 5.5)^2 + (3)(5.42 - 3.5)^2 + (1)(5.42 - 3)^2 + (7)(5.42 - 2.5)^2 + (5)(5.42 - (-0.5))^2$$

$$I_{YY} = 406.9 \text{ ft}^4 = 8.438\ 10^6 \text{ in.}^4$$

Figure 46. Moment of inertia of pile group

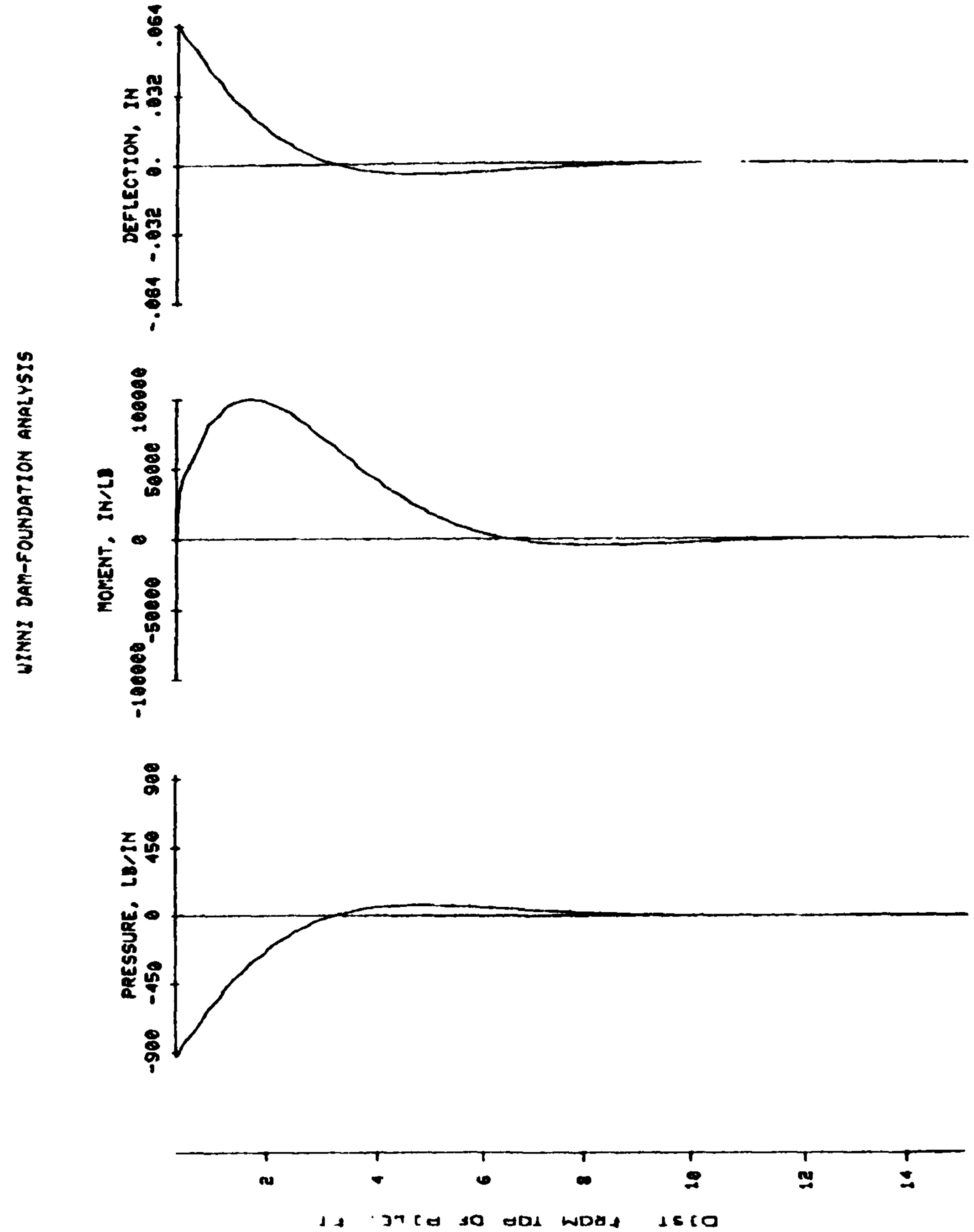


Figure 47. Pressure, moment, and deflection in the most critically loaded pile for normal operation plus ice.

Table 1

General Reservoir Data, Winnibigoshish Dam and Reservoir

Location in miles above Ohio River	1247.9
Located on river	Mississippi
Drainage area (square miles)	1442
Original operating stage limits	1.14 to 15.34* ft
Storage in 1000 acre-ft	976
Present operating stage limits	7.14 to 15.34 ft
Storage in 1000 acre-ft	659
Ordinary operating limits	
Storage in 1000 acre-ft	299
Flowage rights to stage	19.14 ft
Maximum stage of record	15.54 ft
Number of times upper operating limit exceeded	2
Number of times flowage limit exceeded	0
Maximum stage in 1950	15.37 ft
Maximum discharge of record and	4370 sec-ft
year	1905
Elevation of gage zero (U.S.E. datum)	1290.08
Elevation of gage zero: msl 1912 adjustment	1289.47
Elevation of gage zero: msl 1929 adjustment	1288.94
Year of first operation	1884
Normal spring stage drawdown	9.14 ft
Normal summer range	11.14 to 11.64 ft
Desirable bridge clearance, 9.0 ft above reservoir stage of	12.14 ft

* All stages in this table are referred to msl, 1929 adjustment.

Table 2

General Dam Data, Winnibigoshish Dam

Dam	
Type	Earth fill with timber diaphragm core filled with puddled clay
Crest height	23.56* ft
Length	800 ft
Height (maximum)	30.14 ft
Freeboard above maximum stage	8 ft
Control Structure	
Type	Concrete
Sill height	-3.64 ft
Net length of spillway crest	87 ft
Height of piers	16.56 ft
Sluiceways	
Width	14 ft
Number of bays	5
Number of slide gates	5
Number of stop-log sections	15
Height of stop-logs at normal pool	11.64 ft
Log sluice width	12 ft
Fishway width	5 ft (not used)
Discharge channel capacity	2000 sec-ft
Spillway Apron	
Type	Timber
Length	148.5 ft
Width	138.5 ft
Floor height	-5.44 ft
Bridge Over Control Structure	
Floor height	23.56 ft
Roadway width	20.0 ft
Height of walkway	24.56 ft

*All heights or stages are referred to msl, 1929 adjustment.

Table 3

Probe Locations

Hole	Test	Probe Location Below Bottom of Pier (ft)
W-P1A	Test 1	3.99
	Test 2	7.92
	Test 3	11.92
W-P3	Test 1	3.62
	Test 2	7.32
	Test 3	8.47

Table 4

Split Spoon Data - Winnibigoshish Dam

Hole	Depth Into Foundation (ft) From	Depth Into Foundation (ft) To	Blows Per 6-in. Penetration
W-P1A	0.1	0.6	3
	0.6	1.1	7
	1.1	1.6	12
W-P3	1.83	2.33	3
	2.33	2.83	9
	2.83	3.33	10

Table 5

Unconfined Compressive Concrete Strengths

Core Hole	Specimen	Unconfined Compressive Strength (psi)
W-P1	W-P1T	8900
W-P1	W-P1M	9090
W-P1	W-P1B	5070
W-P3	W-P3T	7630
W-P3	W-P3M	6670
W-P3	W-P3M1	5080
W-P3	W-P3B	6890
	Average ≈	7050

Table 6

Patching Material for Cracks, Spalled Joints, and Holes

Material	Parts by Weight
Cement	100
Water	≈18 (adjust as needed)
Acrylic-polymer	27
Fine sand (Passing No. 30 sieve)	150

Table 7

Thin Overlay Material for Concrete Surface Rehabilitation

Material	Parts by Weight
Cement	100
Water	≈20 (adjust as needed)
Acrylic-polymer	30

Table 8

Forces at Top of Pile by Conventional Analysis

Case Loading	Horizontal Load F_H, kips	Number of Piles	Horizontal Load per Pile kips	F_V, kips	e (From Moment Center) ft	Moment about Center of Gravity of Pile Group $F_V(16.26 - e)$, kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force per Pile, kips	Maximum Tensile Force per Pile, kips
Normal operation	287.9	33	8.72	800.86	17.60	-1073	3613	30.1	0
Normal operation with truck loading (H15-44)	287.90	33	8.72	827.5	17.17	-753	3613	29.2	0
Normal operation with earthquake	322.27	33	9.77	800.86	17.20	-753	3613	28.4	0
Normal operation with ice	412.90	33	12.51	800.86	14.68	+1265	3613	31.2	[illegible]
High-water condition	363.7	33	9.53	743.62	16.40	-104	3613	23.1	0

Table 9

Summary of Most Critical Pile Loads and Deflections

Load Case	Pile	Axial Load, kips	Lateral Load, kips	Vertical Deflection at Top of Pile, in.	Horizontal Deflection at Top of Pile, in.
Normal operation	1	30.1	8.7	0.037	0.049
	33	21.7	8.7	0.024	0.049
Normal operation with truck loading (H15-44)	1	29.2	8.7	0.036	0.049
	33	21.7	8.7	0.027	0.049
Normal operation with earthquake	1	28.4	9.8	0.035	0.055
	33	20.9	9.8	0.026	0.055
Normal operation with ice	1	17.4	12.5	0.021	0.071
	33	29.9	12.5	0.037	0.071
High-water condition	1	23.1	11.0	0.028	0.062
	33	22.1	11.0	0.027	0.062

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  Structural stability evaluation Winnibigoshish Dam / by Carl E. Pace (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. ; available from NTIS, 1981.
  26, [43] p. : ill. ; 27 cm. -- (Miscellaneous paper / U.S. Army Engineer Waterways Experiment Station ; SL-81-27)
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  Bibliography: p. 26.
  1.Concrete dams. 2. Piling (Civil engineering). 3. Structural stability. 4. Winnibigoshish Dam (Minn.) I. United States. Army. Corps of Engineers. St. Paul District. II. U.S. Army Engineer Waterways Experiment Station. Structures Laboratory. III. Title IV. Series: Miscellaneous paper (U.S. Army Engineer Waterways Experiment Station) ; SL-81-27.
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